Geotechnical Engineering Laboratory
Soil Mechanics

Contract Report GL-95-XX
February 1996

STRIP DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

by Timothy D. Stark and Thomas A. Williamson

Department of Civil Engineering
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801

Final Report

Approved for public release; distribution is unlimited

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

Waterways Experiment Station Cataloging-in-Publication Data

Strip Drain Test Section in Craney Island Dredged Material Management Area / by Timothy D. Stark and Thomas A. Williamson; prepared for Headquarters, US Army Corps of Engineers.
81 p. : ill. ; 28 cm. -- (Technical report; ITL-95-XX)
Includes bibliographic references.

I. Dredged Material. 2. Soil Densification. 3. Confined Dredged Material. 4. Prefabricated Strip Drains. 5. Service Life of Dredged Material Containment Facilities. II. United States Army Engineer Waterways Experiment Station. III. Geotechnical Engineering Laboratory. IV. Series: Technical report (U.S. Army Engineer Waterways Experiment Station); GL-95-XX. TA7 W34 no. GL-95-XX

CONTENTS

List of Figures

List of Tables

Preface

A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying foundation clay, and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since drains had never been installed in an active dredged material management area, a drain length of approximately 50 m was close to the longest drain ever installed, and the installation equipment had to operate directly on the surface of the soft dredged material. It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future. A supplemental study by the Principal Investigator will address the use of strip drains beneath exterior perimeter dikes to improve existing stability conditions. Preliminary results show that the dredged fill and foundation clay are undergoing substantial (1.8 to 2.5 m in 25 months) consolidation settlement. In summary, prefabricated strip drains appear to be an economical technique for increasing the storage capacity of dredged material management areas.

This research was conducted for the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi and the US Army District, Norfolk (CENAO), Norfolk, Virginia during the period August 1992 to October 1995. The research was performed under Contract Numbers DACW 39-92-M-6666 and DACW39-94-C-0017 between WES and Dr. Timothy D. Stark, an Associate Professor of Civil Engineering at the University of Illinois at Urbana-Champaign. Dr. Stark supervised the research and wrote this report. Mr. Thomas A. Williamson, a Graduate Research Assistant at the University of Illinois at Urbana-Champaign, assisted with the analysis and data reduction.

General supervision in the Geotechnical Laboratory (GL), WES, was provided by Dr. Jack Fowler, Soil Mechanics Division (SMD), Mr. W. Milton Myers, Chief, Engineering Group, SMD, Dr, D.C. Banks, Chief, SMD, and Dr. William F. Marcuson, III, Chief, GL.

General CENAO supervision of the study was carried out by Mr. Sam McGee, under the guidance of Mr. Ronn G. Vann, Chief, Dredging Management Branch, Mr. Thomas C. Friberg, Chief, Operations Section, and Mr. James N. Thomasson, Chief, Engineering Division. Technical information was provided by Mr. Matthew T. Byrne and Ms. Yvonne R. Gibbons, Geotechnical Engineering (GES), CENAO and Mr. David A. Pezza, Chief, GES, CENAO.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was Col. Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic yards	0.76456	cubic meters
cubic fee	0.28317	cubic meters
square feet	0.0929	square meters
square inches	6.452	square meters
feet	0.3048	meters
inches	2.54	centimeters
inches	0.0254	meters
miles	1.60935	kilometers
pounds	4.482	newtons
tons	8.896	kilonewtons
pounds per sqaure foot	47.88	pascals
pounds per square foot	0.04788	kilopascals
pounds per square inch	6.895	kilopascals
tons per square foot	95.76	kilopascals
tons per sqaure foot	0.976	kg/cm^2

I INTRODUCTION

Background

The Craney Island Dredged Material Management Area (CIDMMA) is a man-made 10 km^2 site with a storage area of approximately 8.9 km^2 (Figure 1). Planned in the early 1940's, construction of Craney Island began in August 1954 and was completed in January 1957. Craney Island is the long-term management area for material dredged from the channels and ports in the Hampton Roads area. The CIDMMA is located near Norfolk, Virginia, in Portsmouth, Virginia.

Dredged material has been placed in the management area almost continuously since it was completed in 1957. The original design was for an initial capacity of about 76.4 million m^3 at an annual dredging rate of 3.1 to 5.4 million m^3. Based on an annual dredging rate of 3.8 million m^3, Craney Island was designed for a service life of approximately 20-years (1957 to 1977). Continued dredging in the Norfolk channel has required the capacity of Craney Island to be increased through three major dike raising efforts. The initial dike raising from El. +2.4 to El. +5.2 m Corps of Engineers Mean Low Water (CEMLW) occurred around 1974 with the second increase to El. +7.9 m around 1980. CEMLW is 0.6 m below National Geodetic Vertical Datum 1929, 1972 Adjustment, and 0.2 m below MLW (National Ocean Survey). It should be noted that the water depth at the time of construction was approximately 3.1 m. The US Army Engineer District, Norfolk (CENAO) is currently raising the perimeter dike system based on recommendations presented by Fowler et al. (1987). The west dike is being raised to El. +10.4 m (+34 ft) CEMLW but this raising required the placement of a 305-meter-wide underwater stability berm along the outer toe of the dike (Figure 2) to ensure stability. The perimeter dike in the northwest corner is being raised to El. +10.4 m (+34 ft) CEMLW (Figure 3) using a dike setback of approximately 137.3 m (450 ft). The north and east perimeter dikes are being raised to El. +12.2 m (+40 ft) CEMLW with setbacks from the dike perimeter road of 128.1 m (420 ft) and 137.3 m (450 ft), respectively (Figures 4 and 5). Dike setbacks have resulted in approximately 0.1 km^2 to 0.2 km^2 of lost storage capacity during each dike raising. Figure 1 shows the location of these dike cross sections. After the third raising is completed, the perimeter dikes will be at their maximum height without inducing foundation instability.

Using plans developed by Palermo et al. (1981), interior dikes were built within Craney Island to create three containment areas (Figure 1) that would improve sedimentation in the compartment being filled and allow the other two compartments to desiccate and consolidate at a faster rate. Desiccation will be accelerated by the removal and/or evaporation of surface water,

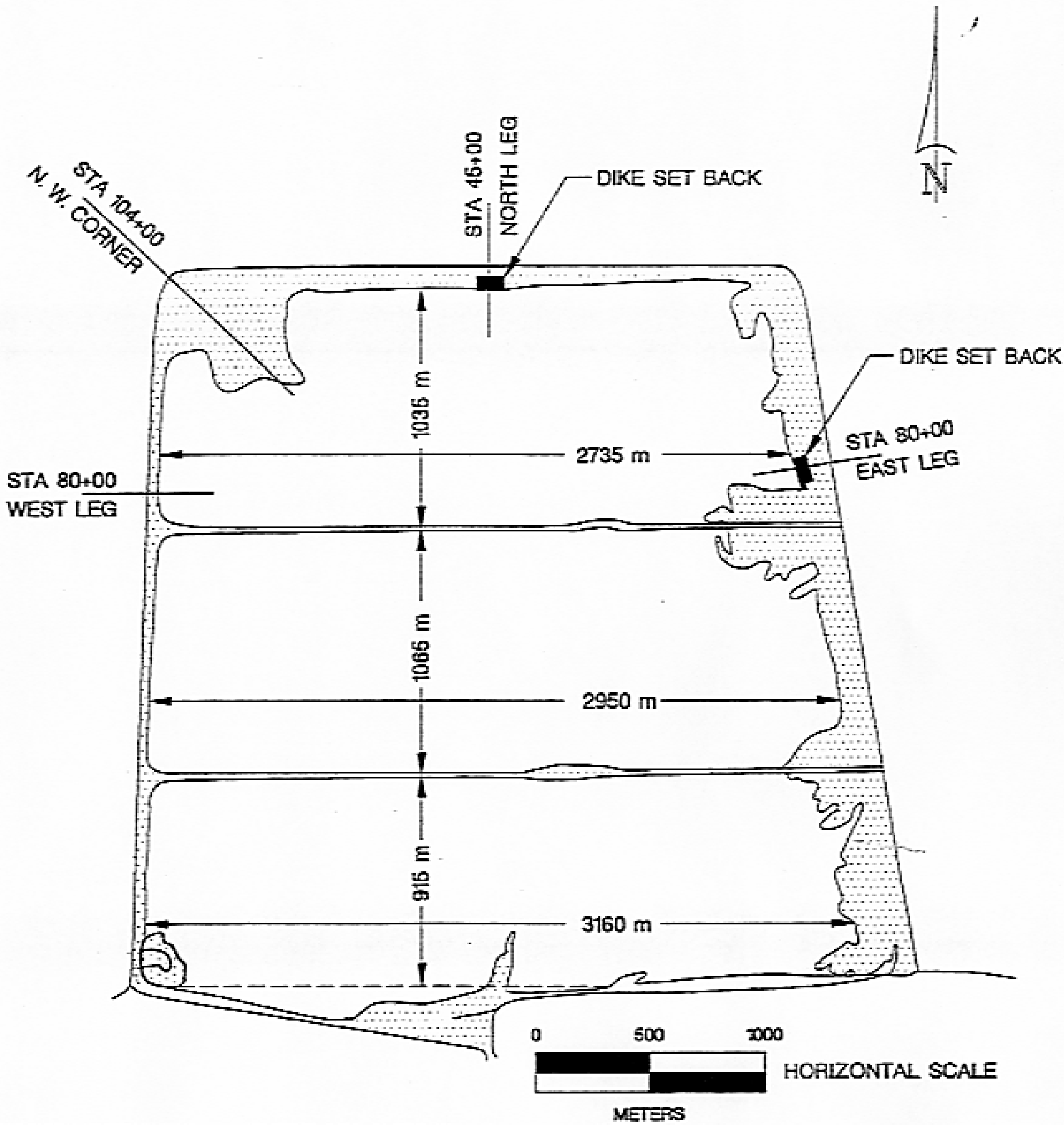

Figure 1. Plan View of Craney Island and Location of Dike Cross Section

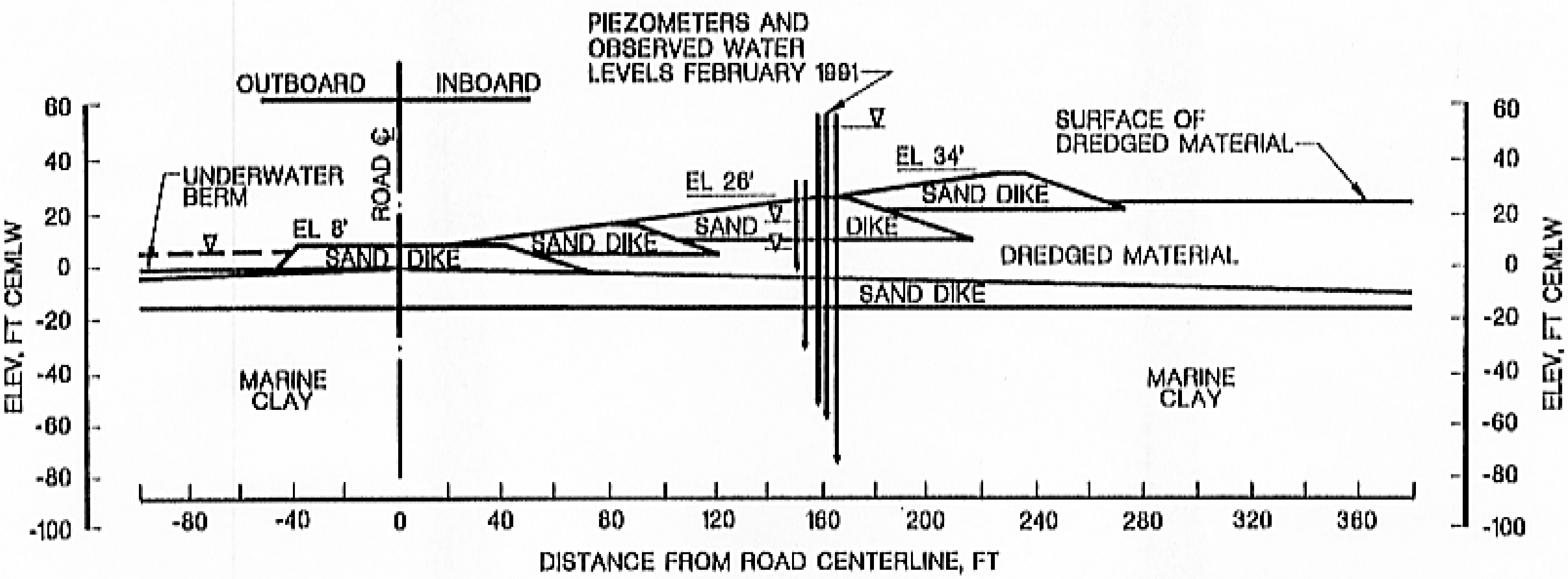

Figure 2. Generalized Cross-Section, West Perimeter Dike (from Fowler et al. 1987)

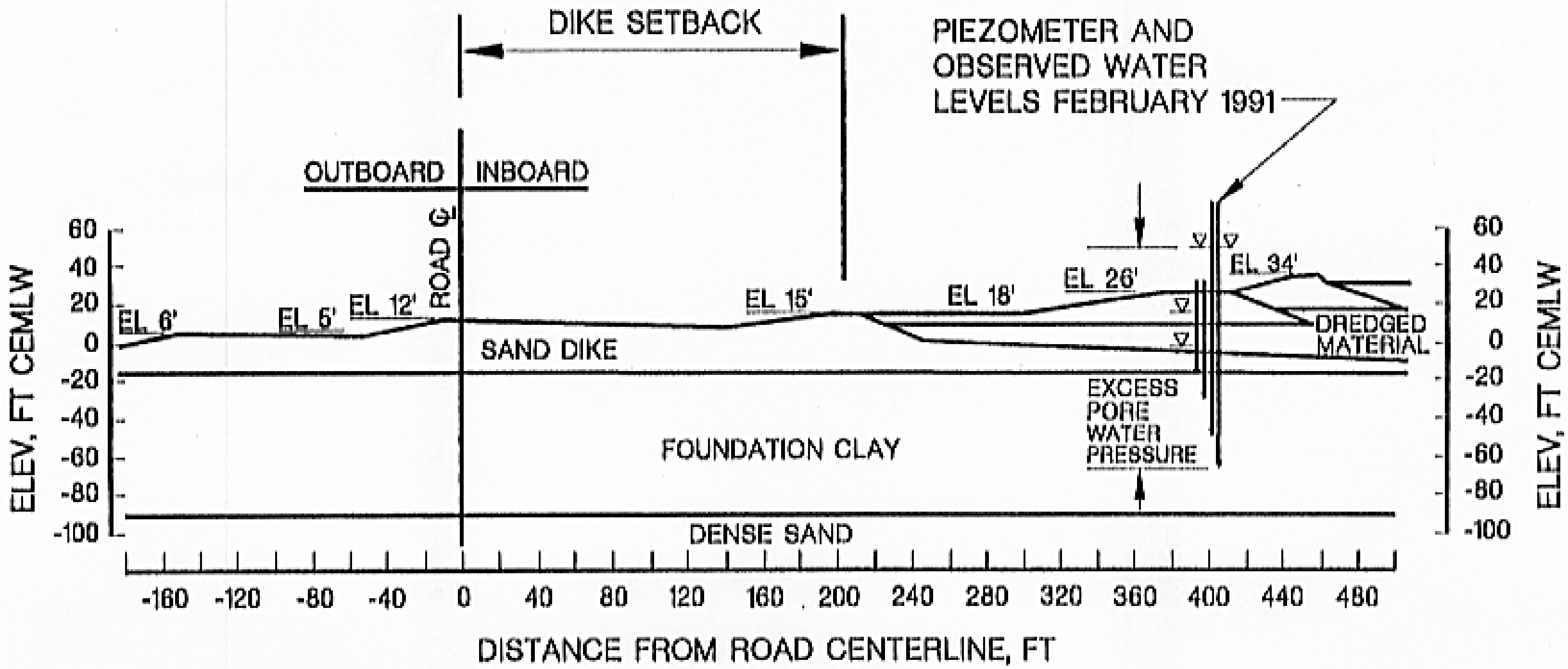

Figure 3. Generalized Cross-Section, Northwest Corner Perimeter Dike (from Fowler et al. 1987)

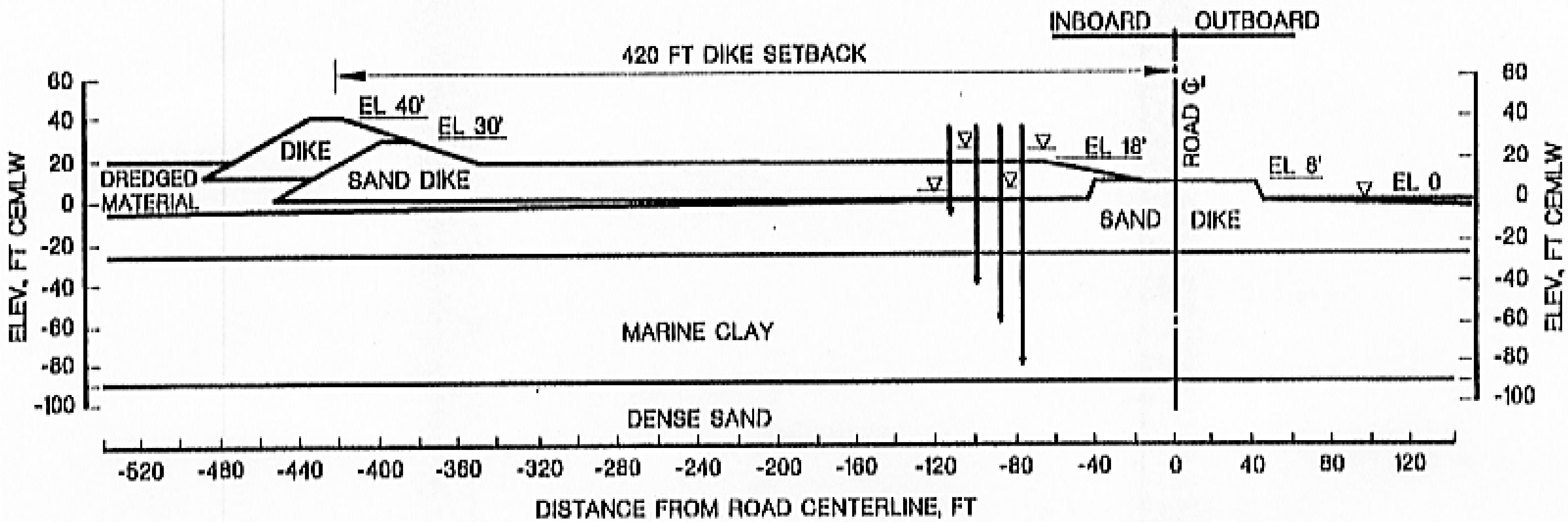

Figure 4. Generalized Cross-Section, North Perimeter Dike (from Fowler et al. 1987)

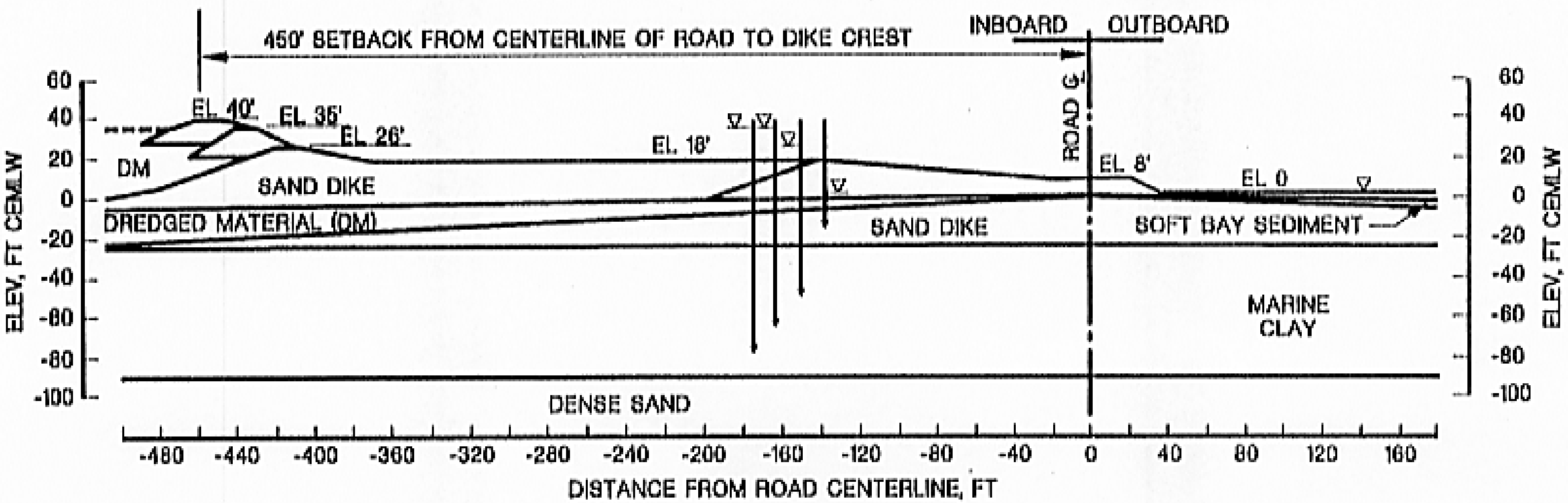

Figure 5. Generalized Cross-Section, East Perimeter Dike (from Fowler et al. 1987)

and will increase the amount of consolidation because the effective unit weight of the soil increases as the pore-water evaporates. Construction of the interior dikes was completed in 1983, and the dredged material management plan (Palermo et al. 1981) was implemented in 1984 starting with the center compartment. The management plan has resulted in each compartment being filled approximately every third year. On the average 3.1 to 3.8 million cubic meters of dredged fill is placed in a compartment each year. This results in an annual increase in dredged fill thickness of 0.9 m to 1.8 m in the compartment being filled. If the site was not subdivided, the annual increase in dredged fill thickness would be approximately 0.3 m to 0.6 m (Szelest 1991).

The Environmental Laboratory at the US Army Engineer Waterways Experiment Station conducted an extensive consolidation and desiccation analysis to predict the remaining service life of the CIDMMA (Palermo and Schaefer 1990). This study utilized the finite strain consolidation microcomputer program PCDDF (Stark 1991; Stark and O'Meara 1991) and revealed that the current capacity of Craney Island will be exhausted around the year 2000. As a result, CENAO began investigating new techniques for increasing the storage capacity of the CIDMMA.

Alternatives for Increasing Storage Capacity at Craney Island

Studies by Fowler et al. (1987) showed that the perimeter dikes are at their maximum height due to the current undrained shear strength of the soft marine clay foundation (Figure 6). However, if the undrained shear strength of the dredged fill and underlying marine clay was increased through consolidation, the perimeter dikes could be raised again. In addition, the increase in shear strength would probably allow the dikes to be raised without setbacks or stabilizing berms. The time required for this consolidation and strength gain is substantial, and thus would not alleviate the short-term storage problem.

An extensive study was conducted by Spigolon and Fowler (1987) on the feasibility of expanding the CIDMMA. Six expansion configurations were considered but the 1991 the Virginia State Legislature ruled that Craney Island could not be expanded or replaced at the present time. Therefore, the feasibility of restricting the usage of the CIDMMA to placement of unsuitable dredged material and ocean placing the suitable material was investigated (Myers et al. 1993). The cost of ocean placement is approximately \$13.0 per m^3 whereas placement in the CIDMMA is about \$1.20 per m^3 (Szelest 1991). CENAO is dredging at a rate of approximately 3.8 million m^3 per yr. Therefore, the difference between placement in the CIDMMA and ocean placement is approximately \$44.8 million per yr. In addition, the environmental impact of ocean placing such a large quantity of dredged material would require substantial study. As a result, additional alternatives for increasing the storage capacity of the CIDMMA were sought.

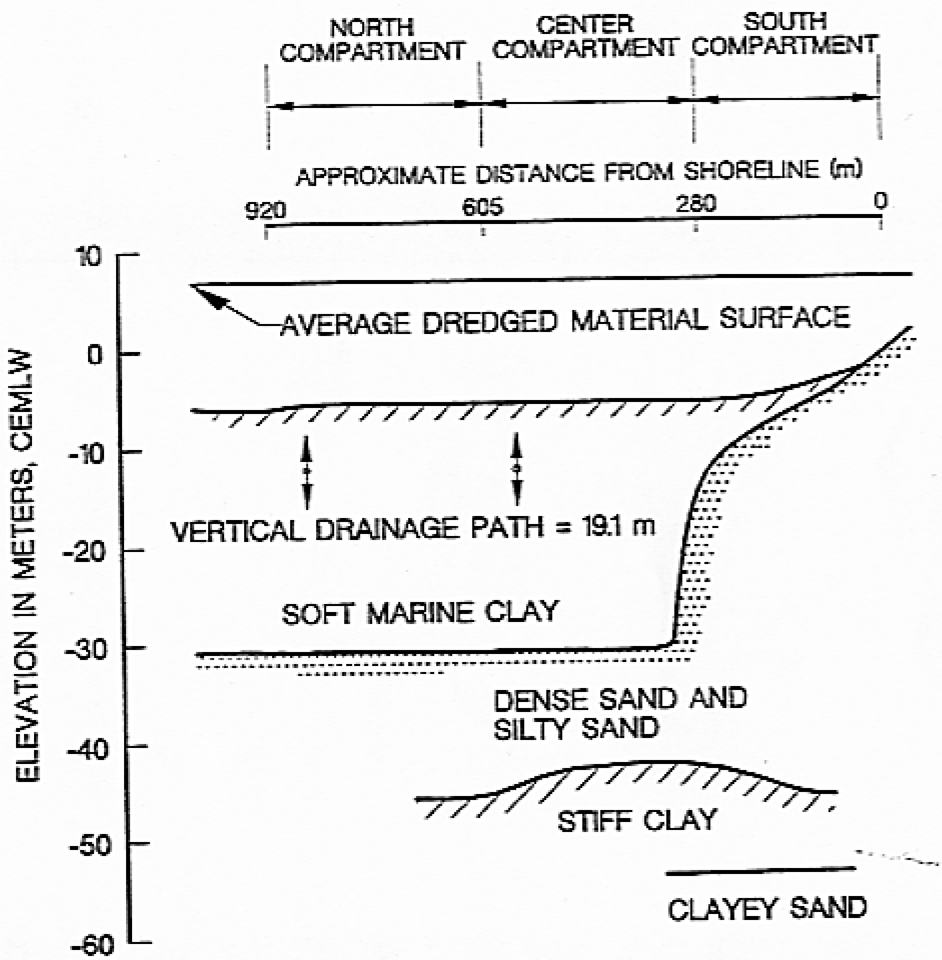

Figure 6. Generalized Subsurface Profile at Craney Island

Recently installed piezometers in the perimeter dikes at the CIDMMA (Stark 1995) revealed that large excess pore-water pressures exist in the marine clay. It can be seen from Figures 2 through 5 that the excess pore-water pressure levels in February 1991 exceed the ground surface elevation by 7.5 m in some locations. The dissipation of these excess pore-water pressures would result in substantial consolidation settlement, and thus increased storage capacity. In addition, consolidation of the marine clay and dredged fill would cause a significant increase in the undrained shear strength of these materials. This would allow the perimeter dikes to be constructed to higher elevations without setbacks or stability berms.

The time required for 90 percent consolidation to occur can be estimated using the one-dimensional consolidation equation (Terzaghi and Peck 1967):

$$t_{90} = \frac{0.848 * H_{dr}^2}{C_v} \tag{1}$$

where H_{dr} is the maximum length of vertical drainage path and C_v is the vertical coefficient of consolidation. This equation shows that the time required for consolidation is controlled by the coefficient of consolidation, that is, permeability, of the soil and the maximum vertical drainage length that water must travel to exit the soil deposit. Since altering the permeability of a soil in situ is not practical, techniques were sought to decrease the drainage path to accelerate consolidation.

Use of Prefabricated Strip Drains to Increase Storage Capacity

Figure 6 shows the generalized subsurface profile at the CIDMMA. It can be seen that the average surface elevation of the dredged fill is +7.3 m CEMLW and the thickness of the dredged fill is about 13.5 m. The thickness of the marine clay is 24.8 m, and thus the combined thickness of the dredged fill and marine clay is 38.2 m. Since the site is doubly drained, the maximum vertical drainage path to either the top surface or the permeable sands underlying the marine clay is 19.1 m. It should be noted that the thickness of the marine clay varies throughout the site. For example, in the north compartment the marine clay is approximately 33.6 m thick, that is to El.-36.6 m, where an old river channel is located. The strip drain test section, described in Part 3 of this report, was located above this old stream channel, and thus the maximum vertical drainage path in this area is approximately 23.6 m. This location was obtained based on the furthest westward reach of a dredge pipe to create the sand blanket in the test section. The channel also dominates the majority of the north compartment. For illustrative purposes the marine clay will be assumed to be 24.8 m thick. Recent piezocone penetration tests in the north compartment described herein suggest that the underlying dense sand and silty sand are permeable, and thus the site is doubly drained.

Figure 7 shows that the installation of vertical strip drains will result in radial flow as well as vertical flow. The strip drains are installed through a 0.6 m sand working platform into the dredged fill and marine clay. The spacing of the strip drains in the test section is 2.2 m, which will be described subsequently. Strip drains reduce the maximum drainage path to one-half of the strip drain spacing, that is, approximately 1.1 m, instead of one-half of the compressible layer thickness, that is, 19.1 m (Figure 6). This reduction in drainage path is extremely significant since the time rate of consolidation is a function of the length of drainage path squared (Equation 1). Therefore, the installation of vertical strip drains will result in a substantial reduction in the time required to consolidate the dredged fill and underlying marine clay. This will yield a rapid increase in storage capacity and undrained shear strength of the dredged fill and marine clay.

It was proposed that strip drains be installed throughout the placement area and subsequently the perimeter dikes (Stark 1995). The strip drains will consolidate the dredged fill and underlying marine clay in the placement area, which may permit future development of this site. Installing strip drains in only the perimeter dikes would be less expensive and may also result in more settlement because of the additional surcharge applied by the dikes. The strip drains would accelerate consolidation of the marine clay underlying the perimeter dikes and allow the dikes to be constructed to higher elevations. However, the strip drains would not consolidate the placement area and thus not reduce the elevation of the placement area. CENAO is interested in consolidating the placement area because it may create opportunities for future development of the site, and possibly the construction of a new placement area. If consolidation of the 8.9 km^2 placement area is not economically feasible, strip drains may be installed in only the perimeter dikes. However, rip-rap and debris, long settled and submerged in the perimeter dikes, may restrict or prohibit installation of strip drains. A supplemental investigation by the Principal Investigator under separate cover will investigate this option.

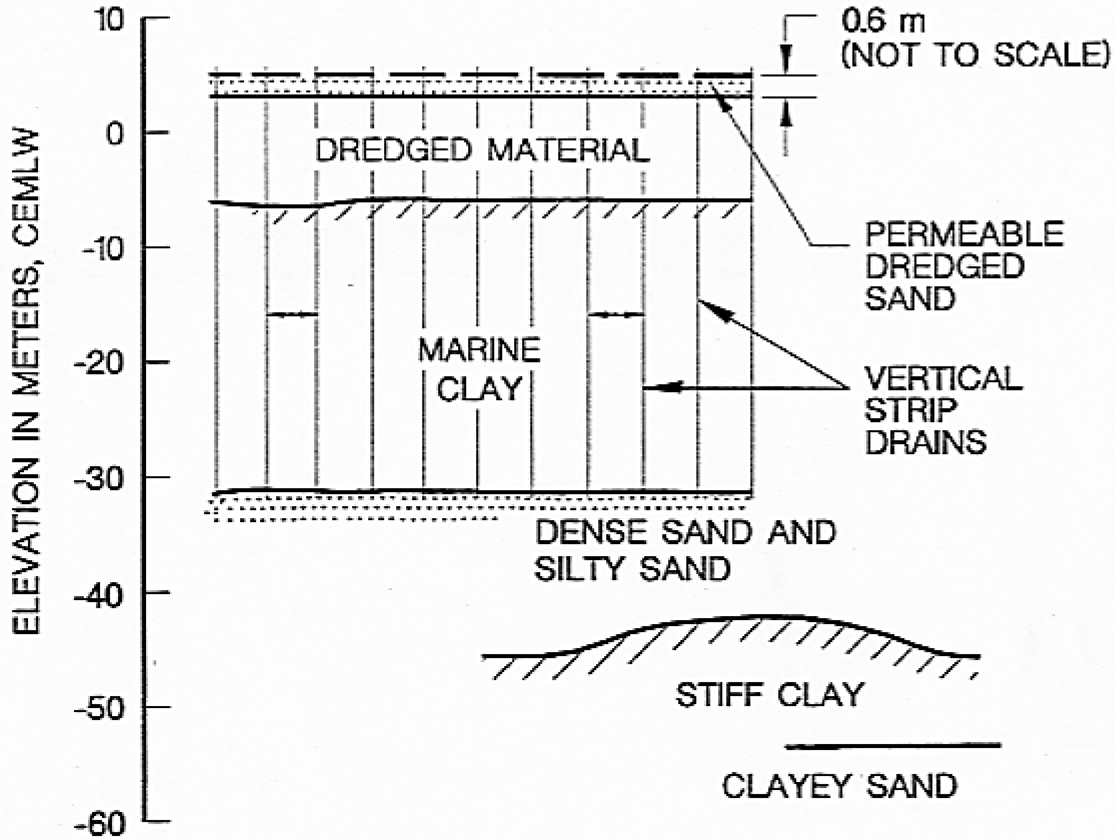

Figure 7. Radial Drainage Pattern Using Vertical Strip Drains

2 VERTICAL STRIP DRAIN TECHNOLOGY

Background

Daniel E. Moran first proposed the use of vertical sand drains as a means for deep soil stabilization in 1925, and he received a U.S. patent on the concept in 1926 (Johnson, 1970). In the last 10 to 15 years, vertical strip drains have replaced conventional sand drains as the preferred method to accelerate the consolidation of soft cohesive soils. This is primarily due to ease of installation, higher flexibility and reliability, less environmental impact, and reduced cost of the strip drains. Most vertical strip drains are modeled after the cardboard strip drain developed by Kjellman in 1948 (a,b). Strip drains are band-shaped and have a rectangular cross section of approximately 10 cm wide and 0.4 cm to 0.5 cm thick. A plastic core with grooves, studs, or channels is surrounded by a filter fabric. The filter fabric is most commonly a nonwoven geotextile. The plastic core carries the excess pore-water to the ground surface and/or the underlying drainage layer, and the filter fabric keeps soils particles from entering the core. Vertical strip drains have been used to accelerate consolidation of soft cohesive soils in many projects throughout the United States, including the recent expansion of the Port of Los Angeles (Jacob et al., 1994), the Seagirt project in Baltimore Harbor (Koerner et al., 1986), the construction of dredged material containment areas in the Delaware River near Wilmington, Delaware (Koerner and Fritzinger, 1988; Fritzinger, 1990), and the New Bedford Superfund site, New Bedford, Massachusetts (Schimelfenyg et al., 1990).

Vertical strip drains are easily installed using equipment (Figure 8) that exerts a ground pressure as low as 20.7 kPa to 34.5 kPa. The installed cost of strip drains is usually \$1.30 to \$3.30 per lineal meter depending on the quantity of strip drains installed. In contrast, the installed cost of conventional sand drains is \$11.50 to \$21.30 per lineal meter. The time required for consolidation of the dredged fill and foundation clay is controlled by the spacing of the strip drains. Therefore, value engineering can be used to determine the optimal spacing of the drains to produce a certain increase in settlement, that is, storage capacity, in a specified time.

The strip drains arrive at the site in large rolls and are installed using a hollow mandrel. The end of the strip drain is threaded down the inside of the mandrel, which must be as long as the depth to which the strip drains are to be installed. At the bottom of the mandrel, the strip drain is threaded through a base plate and the end of the drain is inserted into the mandrel (Figure

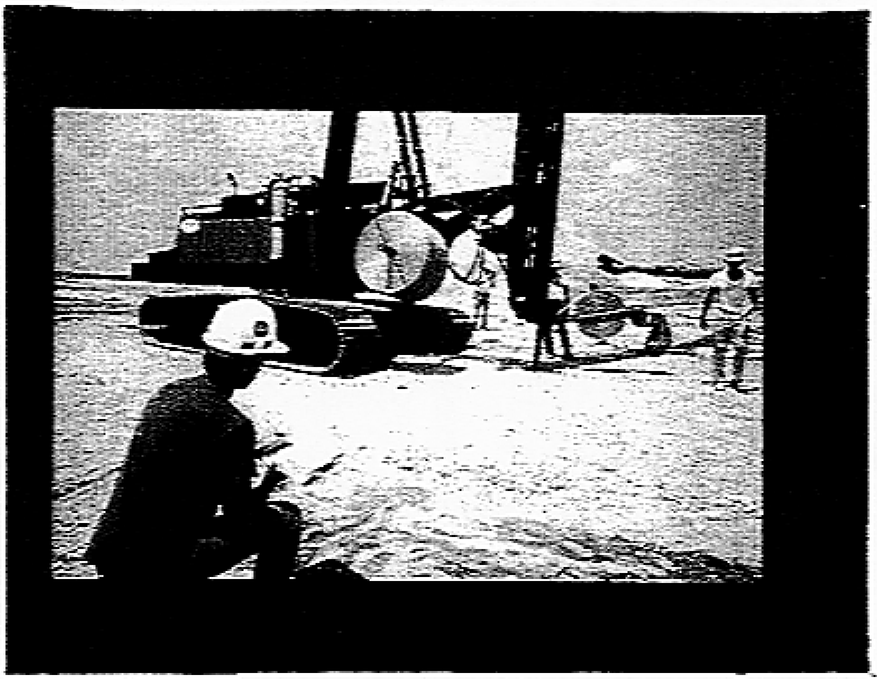

Figure 8. Typical Strip Drain Installation Equipment

9). The base plate is used to keep the strip drain at the bottom of the mandrel during installation, to prevent soil from entering the mandrel during the insertion process, and to keep the strip drain at the desired depth as the mandrel is withdrawn. When the mandrel is withdrawn from the ground, the strip drain is cut, and the process is repeated at the next location. This insertion cycle is rapid (1 to 5 minutes depending on insertion depth) and only strip drains, base plates, and a drain cutting tool are required.

At the CIDMMA it is anticipated that strip drains will be installed in one compartment while the other compartments are used for placement and desiccation. After the strip drains accelerate consolidation in the first compartment, this compartment will be used for placement while strip drains are installed in another compartment and the third compartment undergoes desiccation to support the strip drain equipment. It is anticipated that installation of strip drains will continue until strip drains have been installed in all three compartments.

The length of the strip drains will vary in each compartment with the longest strip drains (approximately 45 to 50 m) being installed in the north compartment where an old river channel is located. A number of contractors, e.g., Joiner (1991), have installed vertical strip drains to similar depths. For example, 36, 40, and 43 meter long strip drains were recently installed in New York, Utah, and Connecticut, respectively (Joiner, 1991). However, strip drains have never been installed in an active dredged material management area. As a result, existing strip drain equipment had to be modified to reduce the ground pressure to less than 10.3 kPa to successfully operate on the soft confined dredged material. In addition, a vertical strip drain length of 45 to 50 m would be close to the longest drain ever installed (less than 60 m). One of the main objectives of the strip drain test section was to investigate the feasibility of installing 45 to 50 m long strip drains from the surface of confined dredged material.

Vertical Strip Drain Design Theories

The design of vertical strip drains is generally based on the theoretical solution for radial consolidation developed by Barron (1948) in which the drains are assumed to be of infinite permeability. Hansbo (1979 and 1981) simplified Barron's solution and accounted for well resistance and the effects of smear caused by drain installation (Figure 10). It can be seen that the degree of consolidation is a function of G and F(n,s). The variable G describes the effect of well resistance on the rate of consolidation and F(n,s) describes the effect of the smear zone. The well resistance is controlled by the influx of water to the strip drain and the flow along the drain. Therefore, G depends on the drain diameter, drain spacing, and the maximum drainage length of the strip drain. When strip drains are installed, the soil adjacent to the drain is disturbed and a

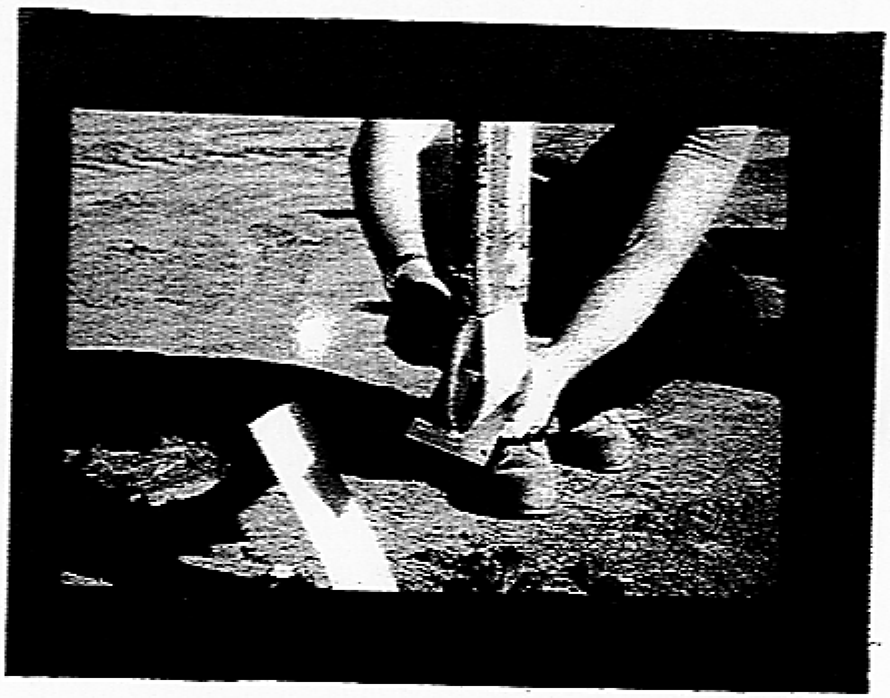

Figure 9. Strip Drain Installation Procedure

Figure 10. Strip Drain Design Theory Presented by Hansbo (1981)

$$U_h = 1 - \exp\left(\frac{-8 * C_h * t}{(d_e)^2 * [F(n,s) \div G]}\right) \qquad (2)$$

$$F(n,s) = \ln\left(\frac{n}{s}\right) + \left(\frac{K_h}{K_s}\right) \ln(s) - 0.75 \qquad (3)$$

$$G = 4\left(\frac{K_h}{K_w}\right)\left(\frac{l_m}{d_w}\right)^2 = \left(\frac{K_h (l_m)^2}{q_w}\right) \qquad (4)$$

where
- U_h = average degree of consolidation for radial flow;
- t = time;
- C_h = horizontal coefficient of consolidation;
- d_e = sphere of influence of the strip drain (triangular pattern = 1.05S where S = strip drain spacing);
- d_w = equivalent strip drain diameter = $\frac{2 * (b \div l)}{\pi}$
- b = width of strip drain (typically 0.305 - 0.328 ft, used 0.31 ft);
- l = thickness of strip drain (typically 0.01 - 0.013 ft, used 0.0115 ft);
- n = ratio of drain diameters = $\frac{d_e}{d_w}$
- $F(n,s)$ = term describing smear zones;
- s = ratio of smear zone diameter to drain diameter = $\frac{d_s}{d_w}$
- d_s = outer radius of the smear zone;
- K_h = horizontal coefficient of permeability of the undisturbed soil;
- K_s = horizontal coefficient of permeability of the smeared soil;
- K_w = coefficient of permeability of the strip drain;
- G = term describing well resistance;
- q_w = discharge capacity of strip drain = $\frac{\pi}{4} d_w^2 K_w$
- l_m = maximum drainage length of strip drain.

Figure 11. Strip Drain Design Theory Presented by Lo (1991)

$$U_h = 1 - \exp\text{-}\left(\frac{8 * C_h}{(d_e)^2 * [F(n,s) + 2.5 * G]} + \frac{4C_v}{H_{dr}^2}\right) t \tag{5}$$

$$F(n,s) = \left(\frac{n^2}{n^2-1}\right) * \left[\ln\left(\frac{n}{s}\right) + \left(\frac{K_h}{K_s}\right)\ln(s) - 0.75\right] + \left(\frac{s^2}{n^2-1}\right)\left[1 - \left(\frac{s^2}{4n^2}\right)\right] + \frac{K_h}{K_s}\left(\frac{1}{n^2-1}\right) * \left[\frac{(s^4-1)}{4n^2} - s^2 + 1\right] \tag{6}$$

$$G = \left(\frac{K_h}{K_w}\right)\left(\frac{l_m}{d_w}\right)^2 = \left(\frac{K_h(l_m)^2}{\frac{4}{\pi}q_w}\right) = \left(\frac{\pi * K_h(l_m)^2}{4q_w}\right) \tag{7}$$

where U_h = average degree of consolidation for radial flow;

t = time;

C_h = horizontal coefficient of consolidation;

d_e = sphere of influence of the strip drain (triangular pattern = 1.05S where S = strip drain spacing);

d_w = equivalent strip drain diameter = $\frac{2 * (b + l)}{\pi}$

b = width of strip drain (typically 0.305 - 0.328 ft, used 0.31 ft);

l = thickness of strip drain (typically 0.01 - 0.013 ft, used 0.0115 ft);

n = ratio of drain diameters = $\frac{d_e}{d_w}$

$F(n,s)$ = term describing smear zones;

s = ratio of smear zone diameter to drain diameter = $\frac{d_s}{d_w}$

d_s = outer radius of the smear zone;

K_h = horizontal coefficient of permeability of the undisturbed soil;

K_s = horizontal coefficient of permeability of the smeared soil;

K_w = coefficient of permeability of the strip drain;

G = term describing well resistance;

q_w = discharge capacity of strip drain = $\frac{\pi}{4} d_w^2 K_w$

l_m = maximum drainage length of strip drain,

H_{dr} = maximum drainage length of vertical drainage path.

3 FIELD TEST SECTION AND SUBSURFACE INVESTIGATION

Field Test Section Objectives and Layout

A 183 m by 122 m field test section was constructed, instrumented, and monitored to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and marine clay in the CIDDMA. The test section was constructed in the north compartment of the CIDMMA because of the presence of a well-developed desiccated crust (Figure 12). The north compartment also requires the longest drains, which will provide a good evaluation of the strip drain equipment and a comparison between measured and predicted effects of smear zone and well resistance. The vertical strip drain test section consists of two areas (Figure 13). The main area is 152 m by 122 m and is covered with a 0.6 meter thick sand blanket to promote surface drainage and support the installation equipment. The vertical strip drains were pushed through the sand blanket to the underlying dense sands (Figure 14). It can be seen that the bottom of the marine clay is located at El. -36.6 m CEMLW because of the presence of an old river channel.

The mobility test section is 30 m by 122 m and utilizes prefabricated horizontal drains to promote surface drainage. The main objective of the adjacent mobility section was to determine whether or not a sand blanket is required to install vertical strip drains throughout the remainder of the management area. As a result, the 15 cm to 30 cm thick desiccated crust in this area must support the installation equipment. The vertical strip drain equipment was required to exert a ground pressure that would enable the equipment to operate on the desiccated crust. It was anticipated that a maximum ground pressure less than or equal to 10.3 kPa would be required to operate on the crust. Installation of strip drains in the test section commenced on 21 December 1992 and terminated on 26 February 1993. The drains were installed in an east to west direction. After the last drain was installed at the west end of the mobility section, the equipment turned-around and returned to the east end of the sand blanket installing strip drains along the way. The equipment repeated this east-west and west-east movement until all of the strip drains were installed.

The vertical strip drains in the mobility section are connected to horizontal strip drains on the ground surface. The horizontal strip drains are 30 cm wide and 2.5 cm thick. Each vertical strip drain is connected to a horizontal strip drain to promote drainage to the surrounding perimeter ditch. Figure 15 illustrates the connection of a vertical strip drain to a horizontal strip

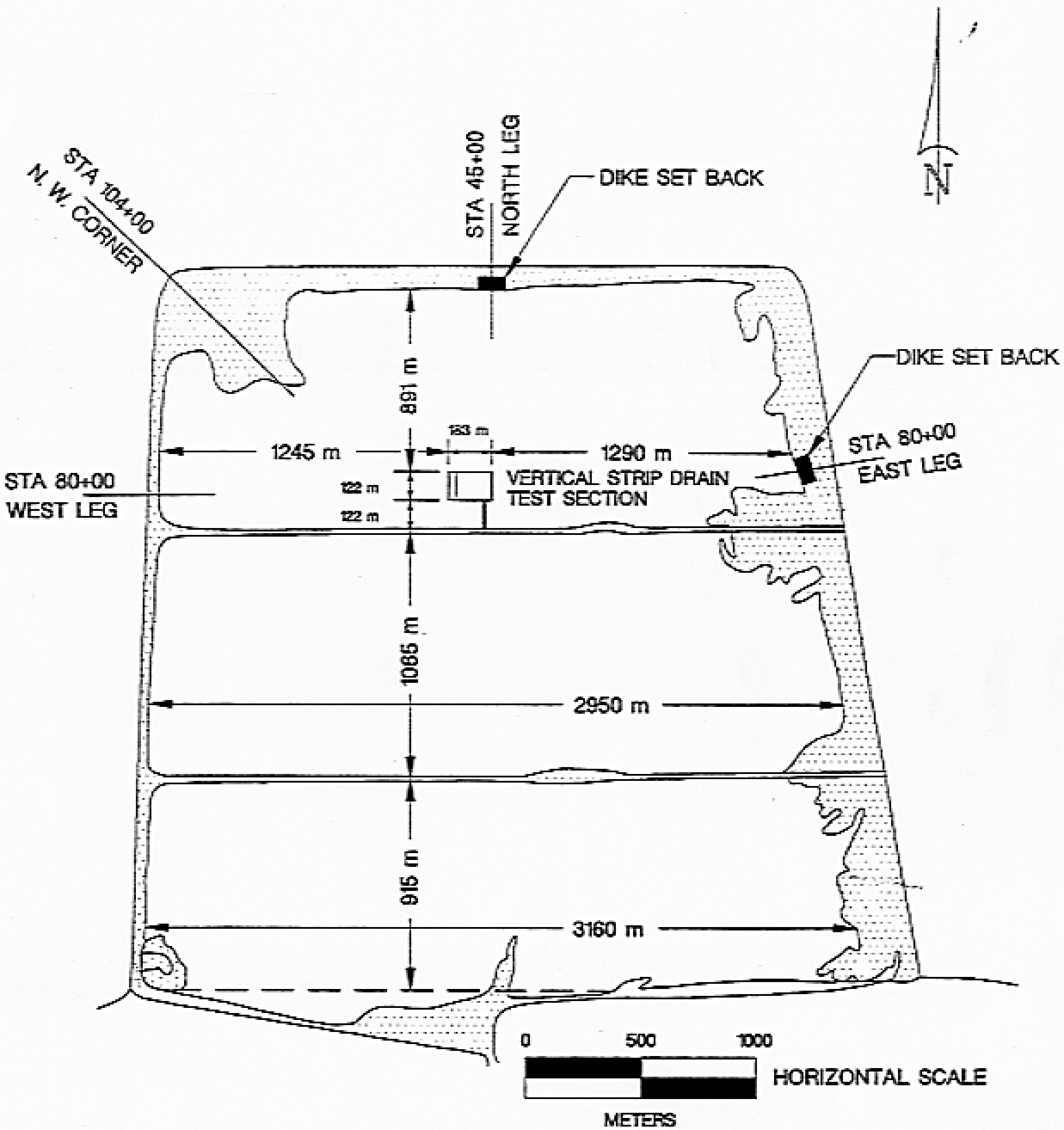

Figure 12. Plan View of Craney Island and Location of Vertical Strip Drain Test Section

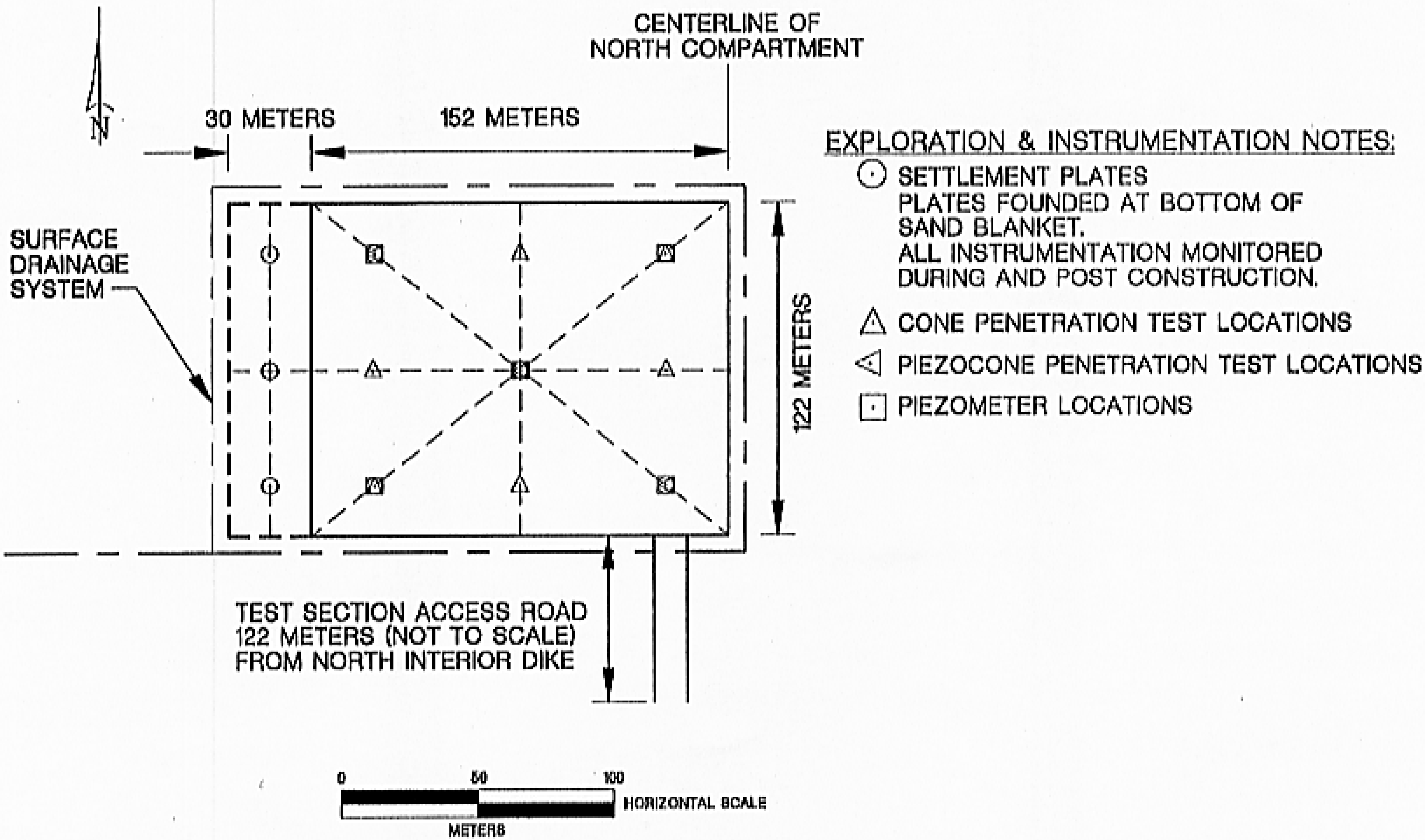

Figure 13. Plan View of Strip Drain Test Section and of Subsurface Exploration and Instrumentation at Craney Island

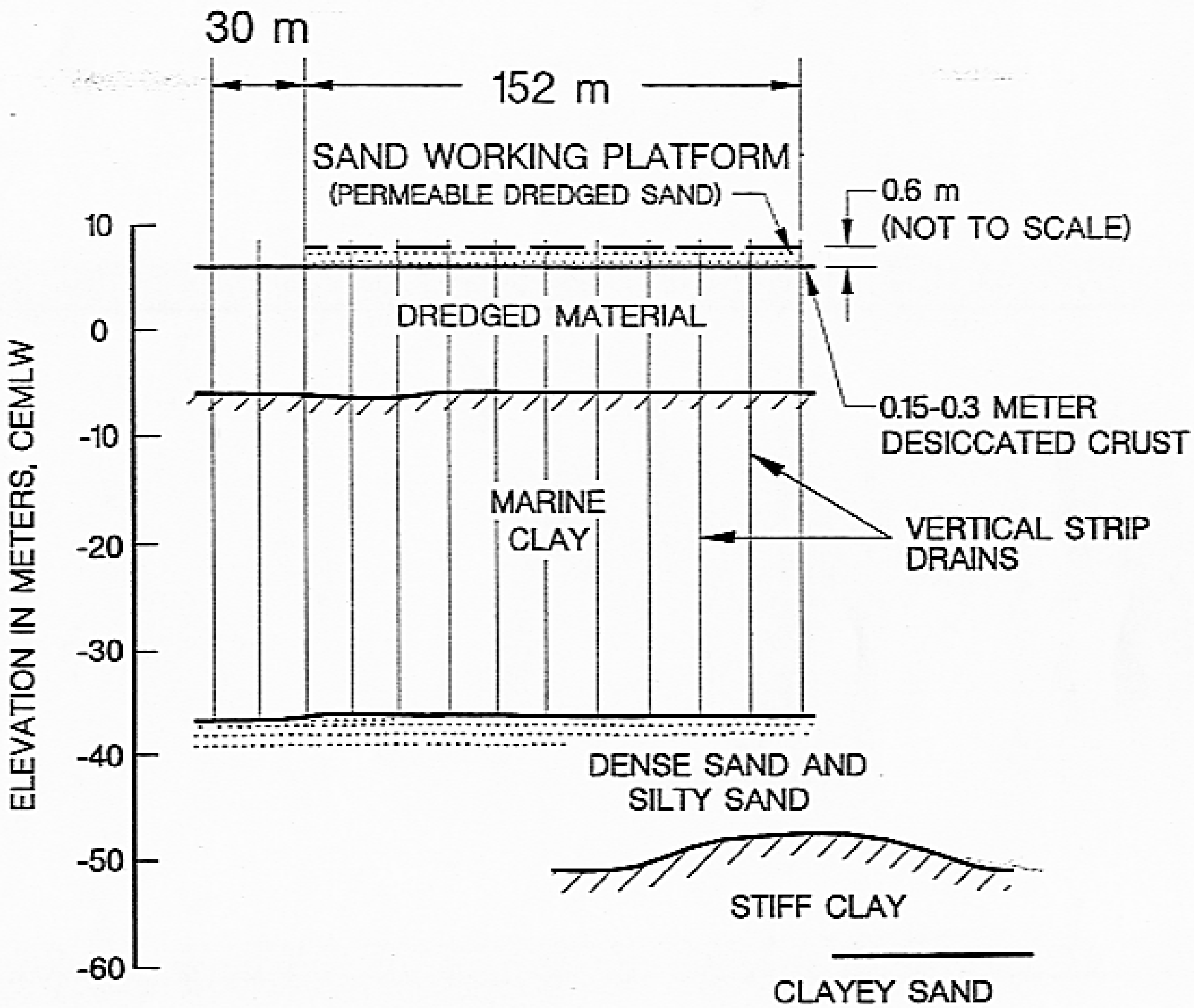

Figure 14. Subsurface Profile Near Vertical Strip Drain Test Section

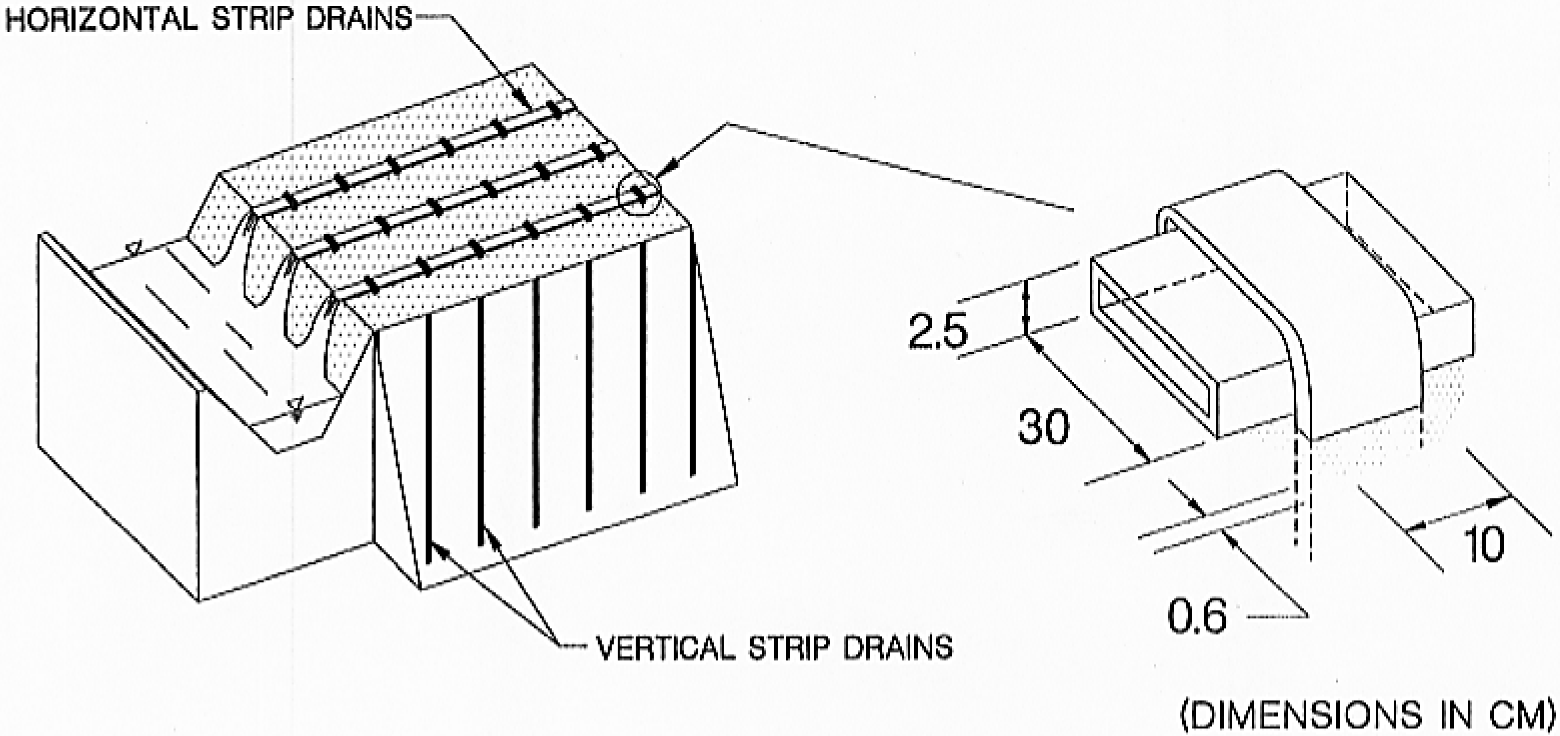

Figure 15. Horizontal and Vertical Strip Drain Installation In Mobility Test Section at Craney Island

drain. Horizontal strip drains are being used to evaluate their effectiveness in conveying water from the test section to the surrounding perimeter trenches and their ease of installation. If the horizontal drains are effective and the desiccated crust can support the installation equipment, a sand blanket would not be required over the remainder of the site. In addition, the horizontal drains will promote drainage as future dredged material is placed in the CIDMMA. Horizontal strip drains were not installed in the main test section because the sand blanket will act as a drainage layer for future dredged fill.

Subsurface Investigation and Field Monitoring

A subsurface investigation was conducted in the test section area before strip drains were installed to aid interpretation of the consolidation settlements. The following is a list of the tests that were conducted to evaluate the subsurface conditions:

1.) Cone and piezocone penetration tests were conducted to define the soil stratigraphy in the test section. These tests results were also used to estimate the magnitude and variability of the undrained shear strength, S_u, and the coefficient of consolidation with depth. Piezocone dissipation tests were used to determine the excess pore-water pressure condition prior to drain installation. Dissipation tests were conducted every 3 m to 6 m in the piezocone tests, and the results were also used to estimate the coefficient of consolidation.

2.) Pneumatic piezometers were installed at three locations (Figure 16) in the test section area using the cone penetration test equipment. A total of eleven piezometers were installed at varying depths at locations P-3, P-5, and P-7 (Figure 16) to aid in determining the variation of pore-water pressure with depth.

3.) Field vane shear tests were conducted every 3 m to 6 m in a boring (B-1) that was drilled at the center of the test section (Figure 16). Water content samples were taken every 3 m in this boring. The moisture content tests were used with the cone penetration test results to estimate the magnitude and variability with depth of S_u, coefficient of consolidation, and initial void ratio.

4.) Eight settlement plates were installed throughout the test section to monitor the effectiveness of the strip drains (Figure 16). Three of the settlement plates are located in

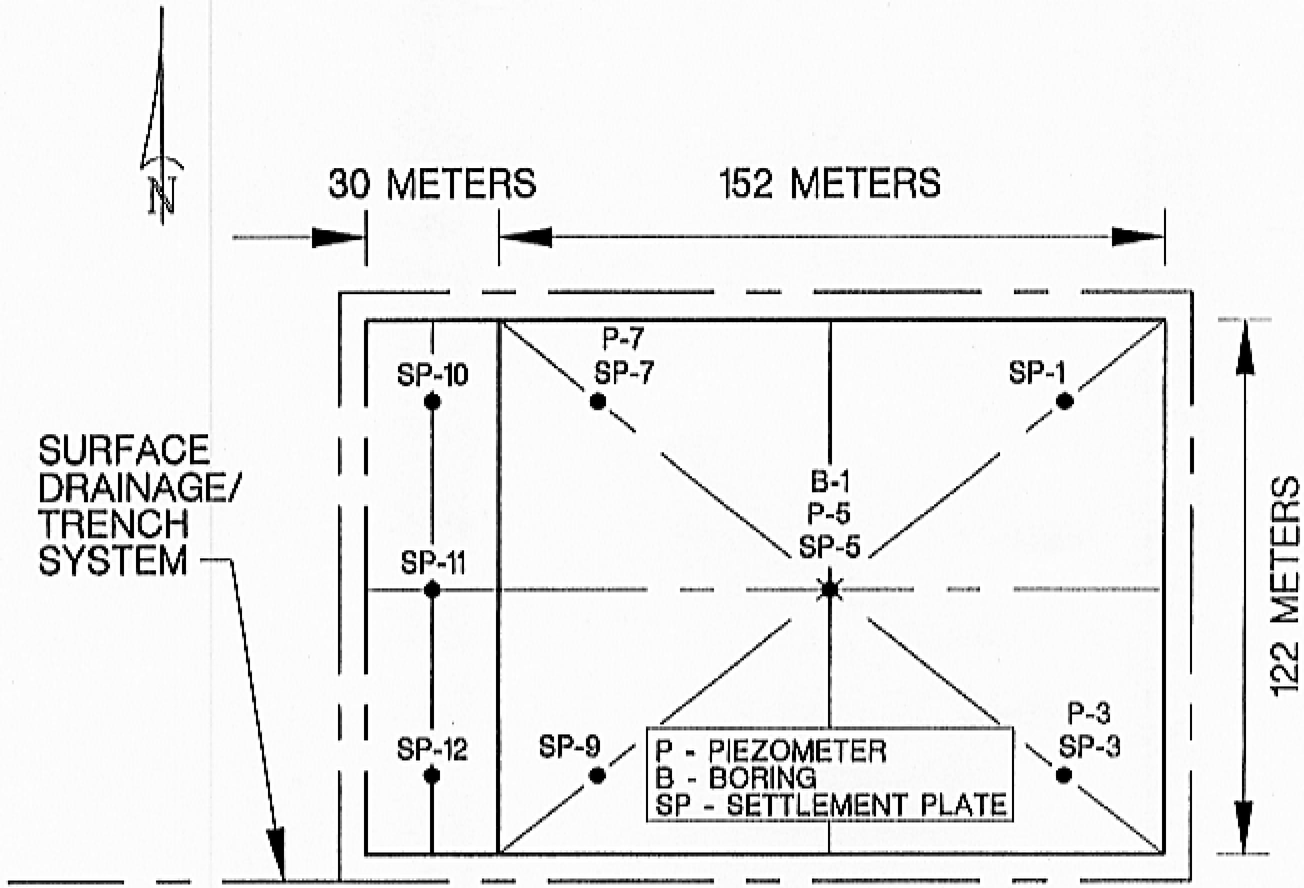

Figure 16. Location of Settlement Plates, Piezometers, and Boring in Test Section

the mobility section and five in the main section. These settlement plates were installed and surveyed prior to installation of the strip drains (18 December 1992). It should be noted that the plates were installed shortly after placement of the sand blanket. The settlement plates were surveyed weekly during installation of the strip drains, which commenced on 21 December 1992. After installation of the strip drains, the plates were surveyed monthly starting with 1 March 1993.

Initial Excess Pore-Water Pressures

Initial excess pore-water pressures were estimated from the installed piezometers and piezocone dissipation tests (Figure 17). These measurements were made prior to strip drain installation. The distribution of excess pore-water pressure clearly indicates that the marine clay is under-consolidated and the underlying dense sand is freely draining. It can be seen that the maximum excess pore-water pressure occurs at a depth between 15 m and 35 m or elevations -7.7 m CEMLW and -27.7 m CEMLW. It can be seen that the excess pore-water pressures at location P-7 (Figure 16) are slightly lower than those at locations P-3 and P-5. It is anticipated that the higher pore-water pressures are caused by the surcharge imposed by the sand blanket and access road near location P-3 (Figure 16). Conversely, P-7 is located near the north edge of the test section where the sand blanket terminates.

The piezocone dissipation tests were conducted until the pore-water pressure measurement was constant. This was monitored using a microcomputer data acquisition system in the testing vehicle. Figure 18 shows the results of a dissipation test conducted at a depth of 27.5 m at the center of the main test area. It can be seen that approximately 80 minutes was required to achieve a constant pore-water pressure. These results are typical of all the piezocone tests, that is, approximately 70 to 80 minutes was required to obtain a constant pore-water pressure. However, plotting the dissipation data on a semi-logarithmic scale (Figure 19) revealed that the degree of consolidation at the end of the dissipation test is less than 90 to 95 percent. As a result, the semi-logarithmic dissipation relationship does not indicate the end of primary consolidation. This prevents the determination of the time at which 100 percent consolidation occurs, and thus the determination of the non-shear induced pore-water pressure.

In summary, the piezocone data in Figure 17 overestimates the excess pore-water pressures because the pore-water pressures generated by cone insertion were not completely dissipated at the end of the test. Further evidence of this is that the effective overburden stress back-calculated using the dissipation test results is negative between depths of 15 m and 35 m. This indicates that the excess pore-water pressures are too high. To ensure 90 to 95%

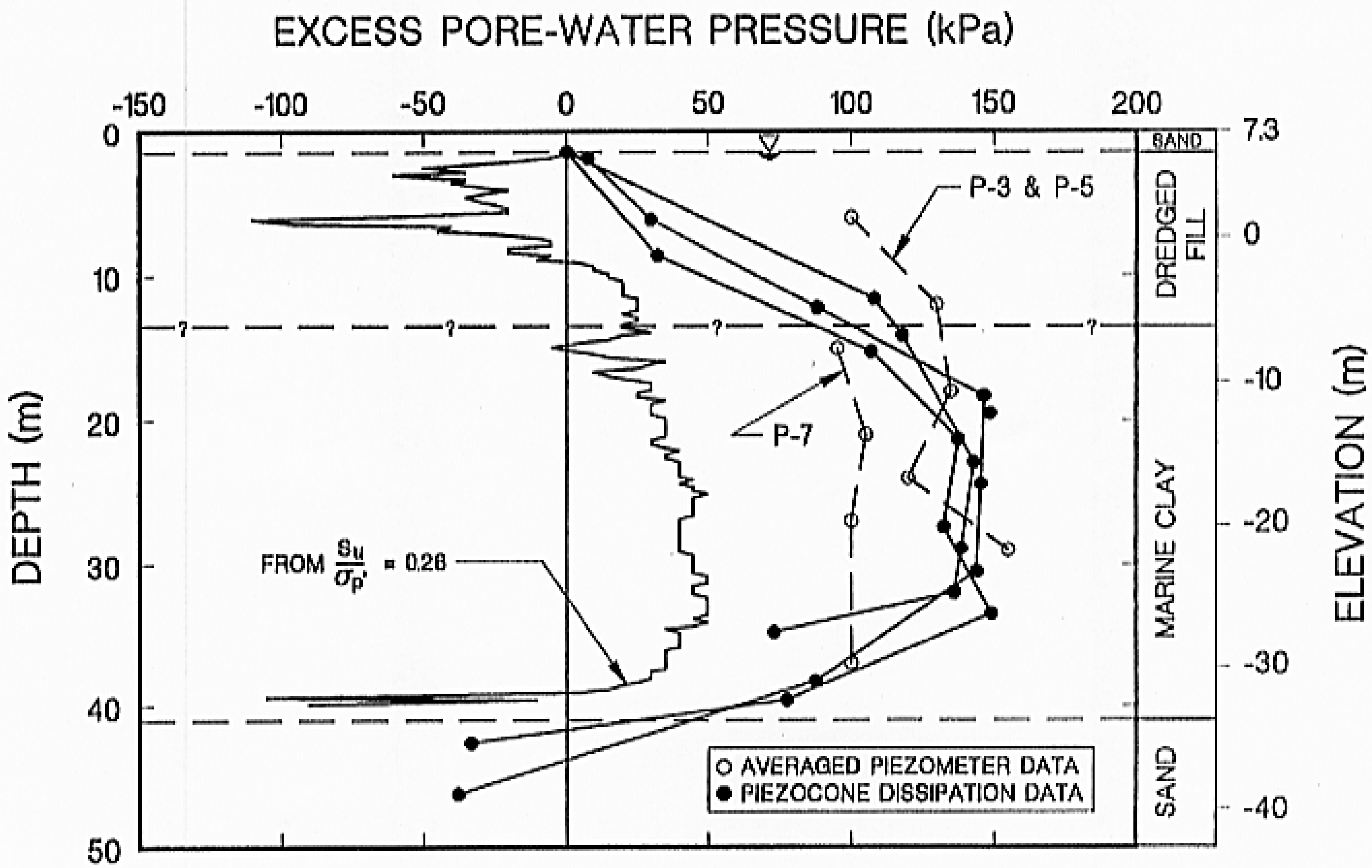

Figure 17. Excess Pore-Water Pressure Under Craney Island Strip Drain Test Section

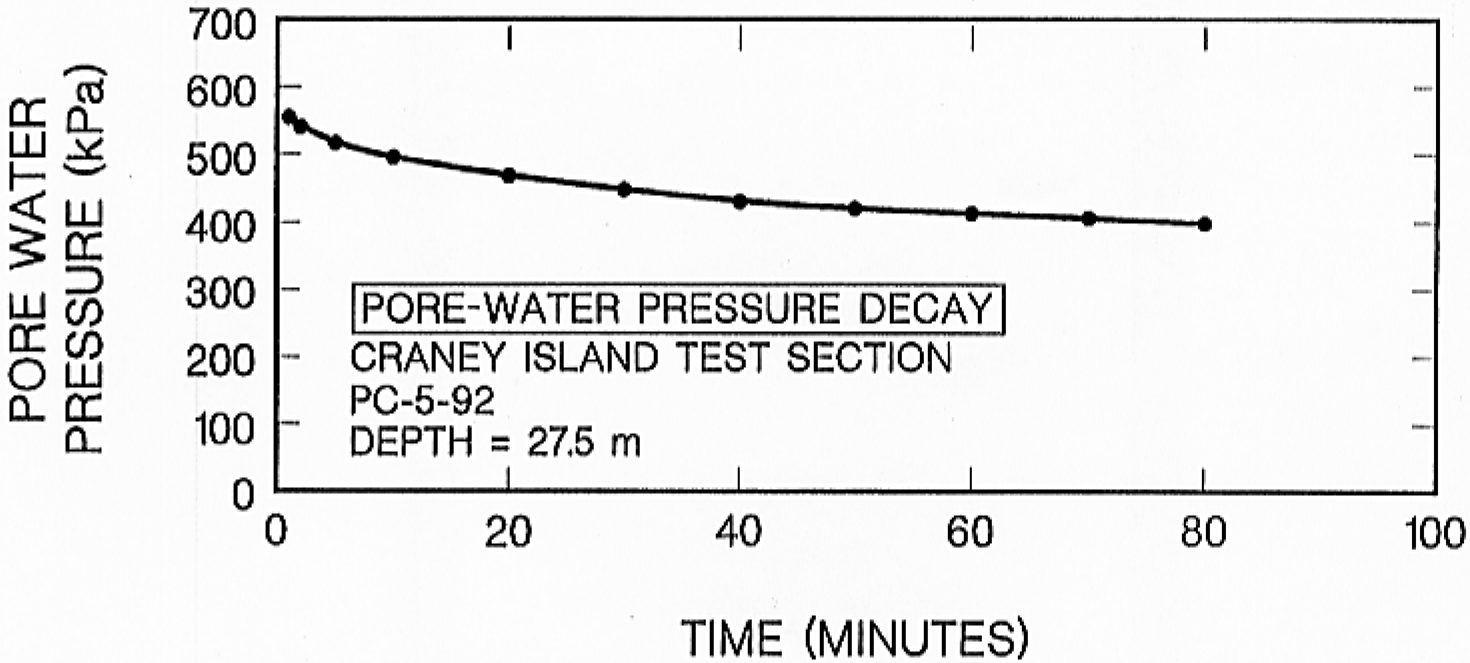

Figure 18. Typical Test Results from Piezocone Penetration Test

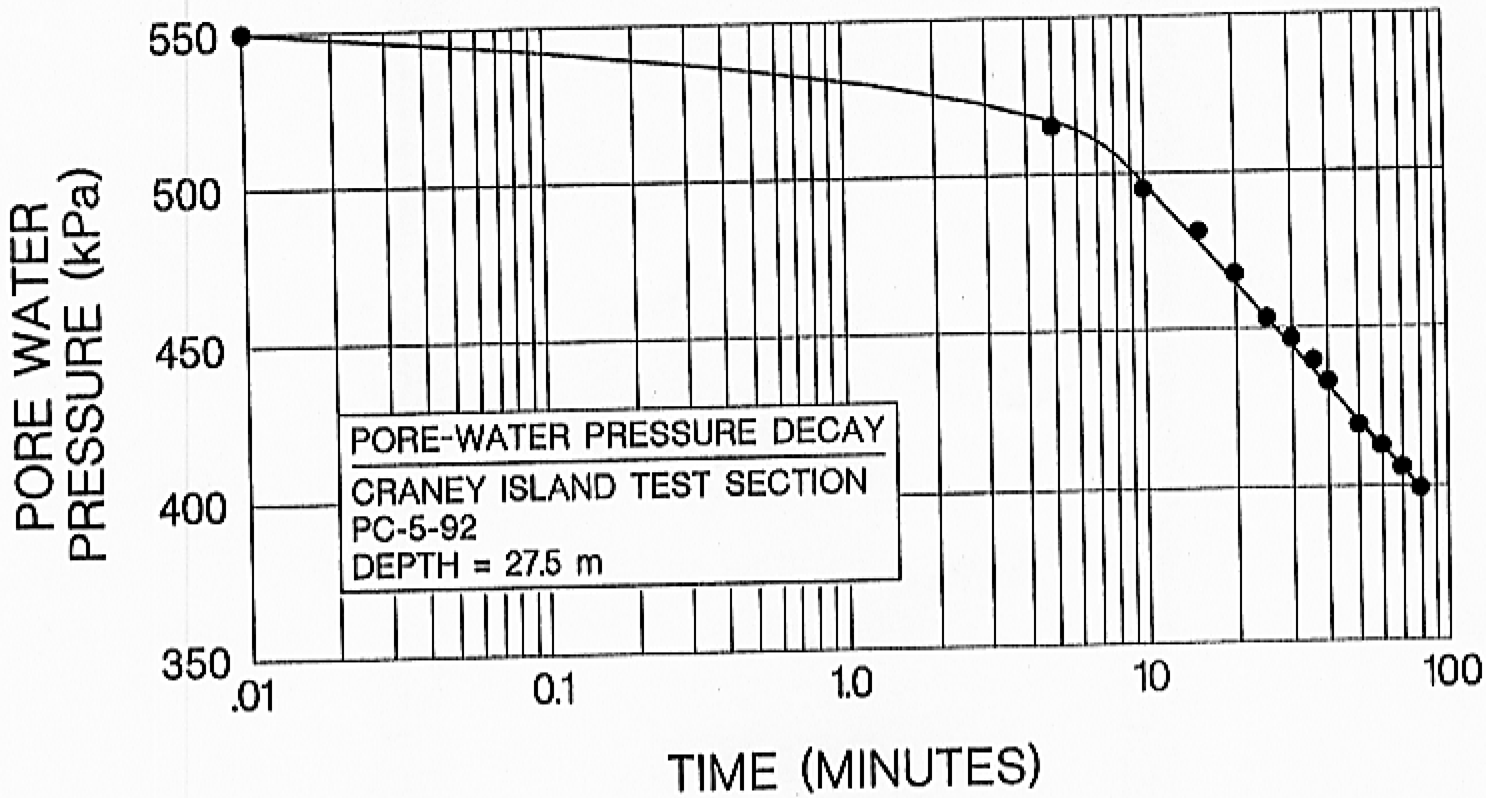

Figure 19. Semi-Logarithmic Presentation of Piezocone Test Results

consolidation is achieved before a dissipation test is terminated, it is recommended that the data acquisition software be modified to present dissipation test results on a semi-logarithmic scale.

To clarify the initial excess pore-water pressure profile additional analytical techniques were utilized. The excess pore-water pressure was estimated from the undrained strength ratio, S_u divided by the preconsolidation pressure, σ_p'. The next section discusses the estimation of the undrained strength and an undrained strength ratio for the dredged fill and marine clay. The preconsolidation pressure was back-calculated by dividing the undrained shear strength and the undrained strength ratio. The preconsolidation pressure was assumed to be equal to the current effective stress because the dredged fill and marine clay are under- or normally-consolidated. The excess pore-water pressure was estimated to be the difference between the current effective overburden stress and the effective stress after 100 percent consolidation. The effective stress after 100 percent consolidation was estimated using a unit weight that corresponds to the void ratio after consolidation (15.4 kN/m^3), the current subsurface stratigraphy, and current elevation of the ground water surface.

It can be seen from Figure 17 that the excess pore-water pressures estimated for an undrained strength ratio equal to 0.26 are less than the piezocone and piezometer data. The undrained shear strength was estimated from the cone tip resistance measured during the cone penetration tests, and thus the profile of excess pore-water pressure is continuous with depth. It should be noted that the piezometers (Figure 17) probably reflect the weight of the sand blanket, and thus show higher excess pore-water pressures than those estimated from the undrained strength ratio. The cone tip resistance does not reflect the sand blanket because the dredged material and marine clay did not have time to consolidate under the stress imposed by the sand blanket. Therefore, the effective stress, cone tip resistance, and undrained shear strength reflect the conditions prior to placement of the sand blanket.

Initial Undrained Shear Strength

Consolidation of the dredged fill and marine clay foundation will result in a rapid increase in storage capacity and soil shear strength. The existing undrained shear strength profile in the test section was estimated using a number of techniques. The first technique described utilizes the tip resistance from cone penetration tests and the following equation:

$$S_u = \frac{q_c - \sigma_{vo}}{N_k} \qquad (8)$$

where q_c is the cone tip resistance, σ_{vo} is the total vertical overburden pressure, and N_k is an empirical cone factor based on field vane shear tests. Empirical correlations of N_k have been developed using the results of field vane shear tests (Lunne and Kleven 1981 and Meigh 1987) and unconsolidated-undrained triaxial tests (Stark and Delashaw 1990). To differentiate the unconsolidated-undrained triaxial mode of failure from the vane shear test, Stark and Delashaw (1990) denoted their cone factor N_{kuu}. Both correlations utilize plasticity index (PI) to estimate values of cone factor.

Table 1 presents the index properties of the marine clay at Craney Island. The statistical values of the index properties were determined from the results of 135 tests. Since the dredged material is similar to the foundation clay the same index properties were used for both deposits.

The field vane value of N_k was estimated for a PI of 41 ranges from 10 to 15, while the value of N_{kuu} ranges from 8 to 14. Since field vane shear test data are not available to estimate a site-specific N_k value, an average value of N_k equal to 12 was utilized in the analysis to facilitate comparison purposes. In addition, the average value of N_k is only slightly higher than the average N_{kuu} for this plasticity index and the unconsolidated-undrained triaxial mode of failure is more suited to stability analyses involving soft saturated clays than the uncorrected field vane shear test. Figure 20 presents the variation of undrained shear strength with depth using N_k equal to 12. Each data point corresponds to a calculation of S_u using Equation (8), the appropriate total stress, and a value of N_k equal to 12.

Table 1. Summary of Index Properties of Marine Clay (after Ishibashi et al. 1993)

	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Clay Size Fraction (%)	Specific Gravity of Solids
AVERAGE	70.2	70.7	29.3	41.4	94.4	2.71
STANDARD DEVIATION	12.4	14.7	4.88	12.3	7.25	0.04
COEFFICIENT OF VARIATION	0.17	0.21	0.17	0.3	0.04	0.02

Figure 20 presents the variation in S_u with depth estimated from cone penetration test results and several interesting facts can be ascertained from the profile. First, the dredged material contains many sand and/or silt seams. This explains the lack of large excess pore-water pressures measured in the piezocone dissipation tests and piezometers in the dredged fill. The dredged fill is probably undergoing self-weight consolidation and the excess pore-water pressures

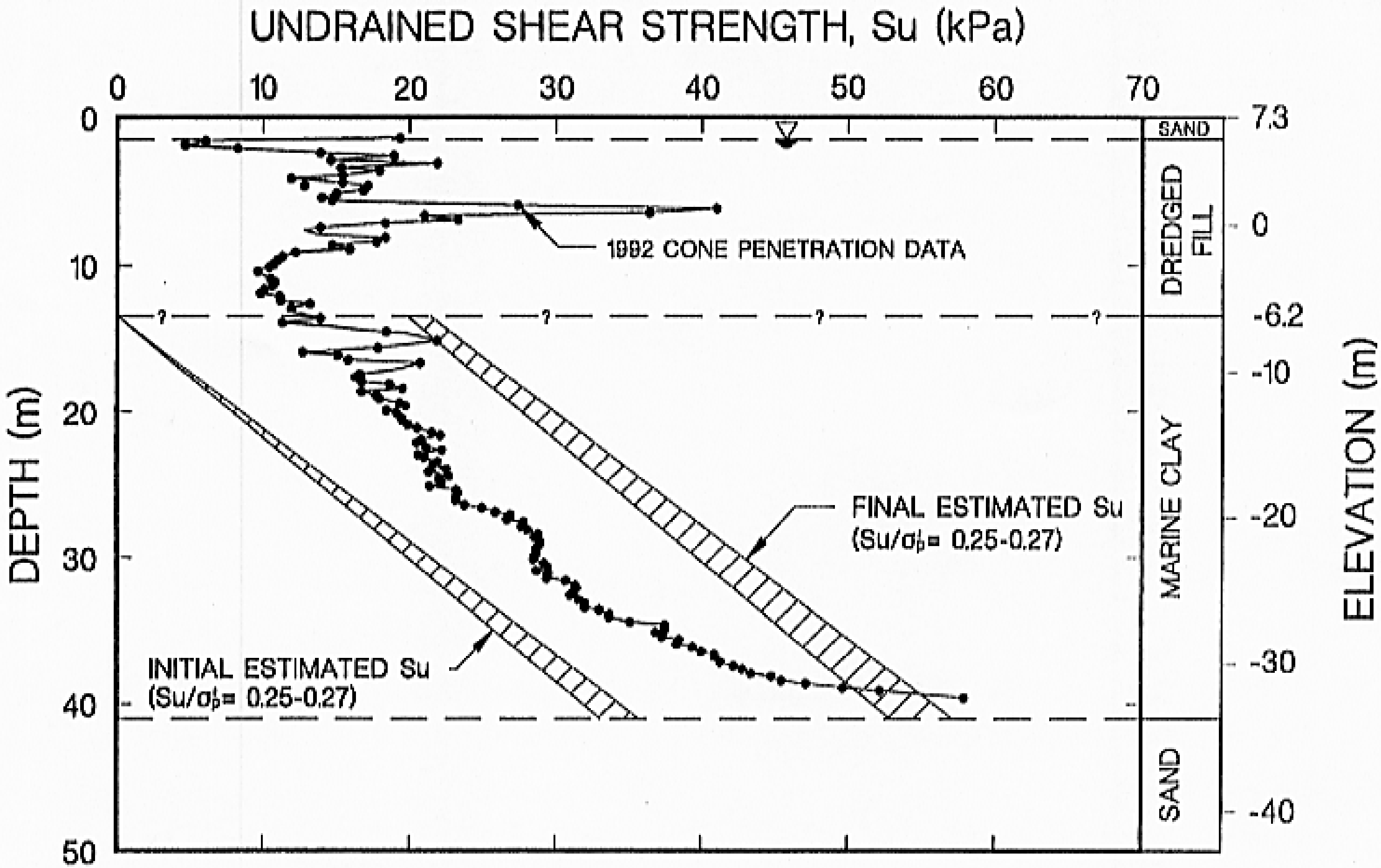

Figure 20. Undrained Shear Strength versus Depth in Craney Island Strip Drain Test Section

are being dissipated by the sand/silt seams. Based on this conclusion, the majority of the settlement measured in the test section is attributed to consolidation of the marine clay. The dredged fill appears to be undergoing self-weight consolidation and acting as a surcharge for the marine clay.

Secondly, the marine clay appears to be under- or normally-consolidated. This is evident by the smoothness of the S_u profile and slight increase in S_u with depth. In addition, it appears that the sand underlying the marine clay is free-draining because the values of S_u increase near the bottom of the marine clay. In fact, the value of S_u near the bottom of the marine clay corresponds to the effective stress at 100 percent consolidation.

Thirdly, the interface between the dredged fill and marine clay appears to be located at a depth of approximately 13.5 m or El. -6.2 m CEMLW. Craney Island was constructed in approximately 3 to 4 m of water. Therefore, it appears that the dredged fill and marine clay interface has subsided 2.2 m to 3.2 m since 1957.

Undrained Shear Strength Ratio

The undrained strength ratio of the marine clay was estimated from historic field vane shear (FV), unconsolidated-undrained triaxial (UU), unconfined compression (UC), and isotropically consolidated-undrained (CU) triaxial tests collected since 1948. Table 2 summarizes the values of undrained strength ratio (S_u/σ_p') estimated from the tests reported in the General Design Memorandums (U.S. Army 1949 and 1986) for Craney Island. Table 2 reveals that the undrained strength ratio ranges from 0.24 to 0.28. The presence of gas in the dredged fill and marine clay complicates the collection and testing of undisturbed specimens. As a result, the most reliable measure of the in-situ undrained strength ratio is obtained using a field test, such as the field vane shear or cone penetration test. It can be seen that the historic field vane shear data yield an average undrained strength ratio of 0.26.

For comparison purposes, the undrained strength ratio was estimated from published correlations, such as Figure 21 (a). The undrained strength ratio for an average plasticity index of 41 and the field vane mode of shear ranges from 0.25 to 0.27 with an average of approximately 0.26. The triaxial compression mode of shear yields an undrained strength ratio of approximately 0.31. Mesri (1989) recommends an undrained strength ratio of 0.22 for stability analyses because a circular failure surface involves the following three modes of shear: triaxial compression, direct simple shear, and triaxial extension (Figure 21b). A value of undrained strength ratio of 0.22 represents an average triaxial compression, direct simple shear, and triaxial extension modes of shear based on the applicable length of the circular failure surface. Since this report is describing

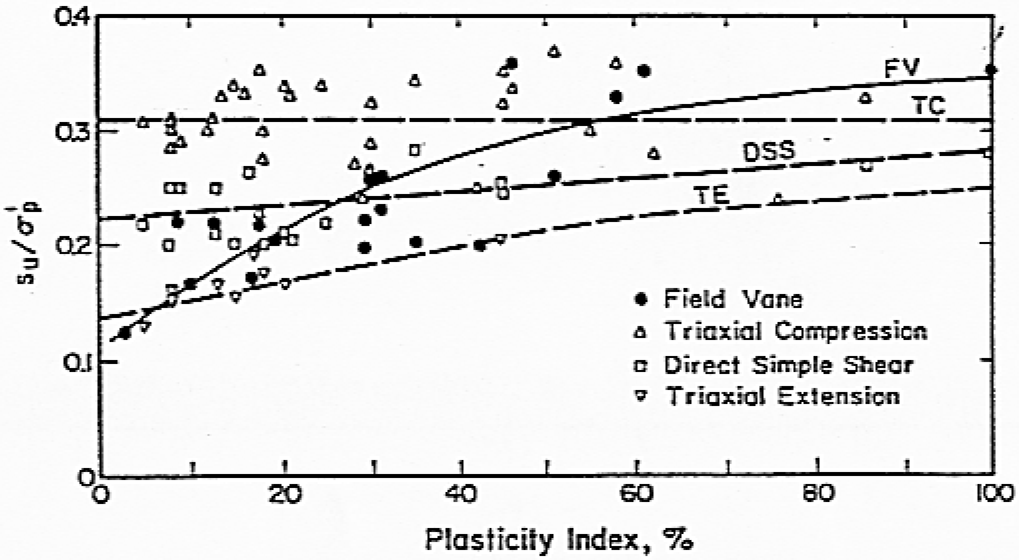

Fig. 21 (a). Values of Undrained Strength Ratio from Laboratory and Field Tests (after Bjerrum 1972)

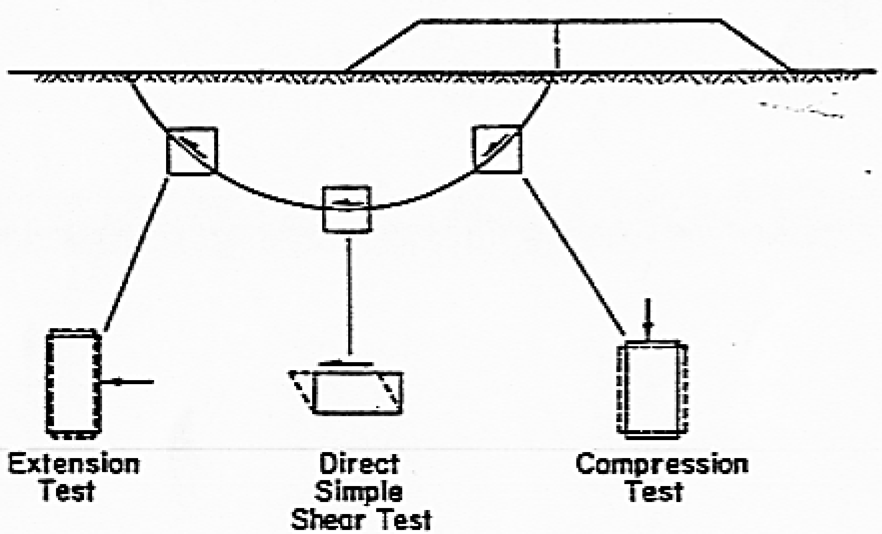

Fig. 21 (b). Modes of Shear Encountered in the Field (Jamiolkowski et al. 1985) the undrained strength gain below the test section and not the undrained shear strength for a slope stability analysis, a range of undrained strength ratio of 0.25 to 0.27 was used in Figure 20 and an average ratio of 0.26 was used in the analysis described herein. However, an undrained strength ratio at or near 0.22 should be considered when evaluating the stability of the perimeter dikes, which usually involves a circular failure surface.

Table 2. Undrained Strength Ratios for Marine Clay from Various Test Methods (after Ishibashi et al. 1993)

TEST METHOD	NO. OF MEASUREMENTS	AVERAGE S_u/σ_p'	STANDARD DEVIATION	COEFFICIENT OF VARIATION
FV	102	0.26	0.04	0.16
UU	55	0.24 *	0.13	0.46
UC	56	0.28	0.16	0.55
CU	10	0.27	0.05	0.17

* Values of Undrained Strength Ratio Higher than 0.7 were Omitted

The undrained strength ratio was used to estimate the S_u profile using a measured unit weight of 14.6 kN/m^3 (Ishibashi et al. 1993) and by assuming that the marine clay is saturated and normally consolidated. Figure 20 shows the initial estimated S_u profile, which corresponds to the S_u profile before Craney Island was constructed, that is, prior to 1956. As a result, the range of S_u was estimated using an S_u/σ_p' equal to 0.25 to 0.27 and a normally consolidated marine clay starting at an original mudline of El. -6.2 m CEMLW. A comparison of the initial estimated profile and the profile estimated from the 1992 cone penetration tests reveals that a small amount of consolidation, and thus shear strength increase, has occurred in the marine clay between 1956 and 1992 for depth ranging from 25m to 35 m.

Undrained strength ratios of 0.25 and 0.27 were also used to estimate the increase in S_u that will result from installation of prefabricated strip drains, and thus 100 percent consolidation of the marine clay. Figure 20 shows the final estimated S_u profile, which was estimated assuming the dredged fill and marine clay are normally consolidated and the dredged fill surface is at El. +7.3 m CEMLW. A unit weight of 15.4 kN/m^3 was used to estimate the undrained shear strength after 100 percent consolidation. A unit weight of 15.4 kN\m^3 was estimated using an average void ratio after consolidation of 2.0 and a degree of consolidation of 100 percent. This value is greater than the initial value of 14.6 kN/m^3 reported by Ishibashi et al. (1993) and reflects the densification by consolidation.

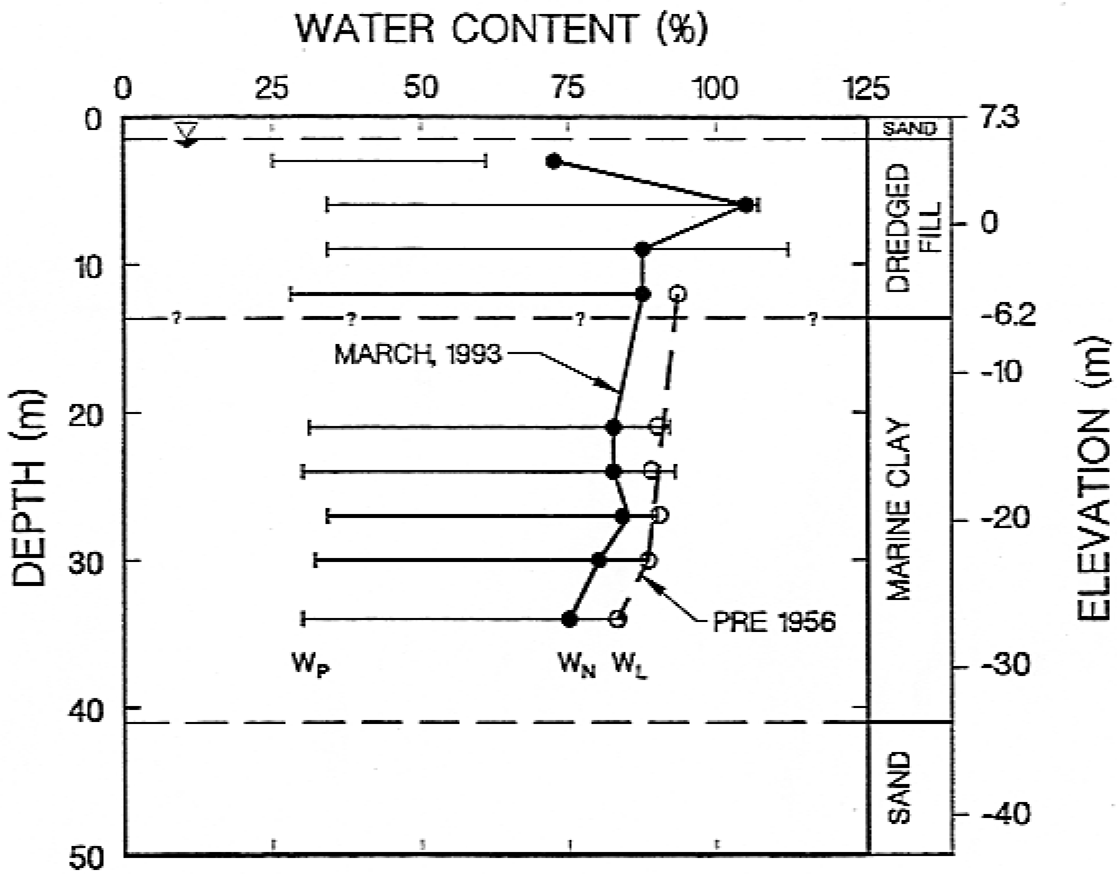

Figure 22. Water Content versus Depth in the Craney Island Strip Drain Test Section

Initial Void Ratio Profile

Void ratios were determined for the samples obtained from the boring at the center of the test section (Figure 16). The void ratios (e) were estimated using a degree of saturation (S) of 100 percent, a specific gravity of soil mass (G_S) equal to 2.71 (Table 1), and the following equation:

$$S * e = G_s * W_N \tag{9}$$

It can be seen from Figure 23 that the average void ratio of the marine clay is approximately 2.5. The dredged fill exhibited void ratios of 2 to 3 and considerably more scatter than the marine clay. Figure 23 also presents a pre-1956 void ratio profile estimated from the water content test results (Figure 22) obtained from the pre-construction subsurface exploration (U.S. Army 1949). It can be seen that the void ratio of the marine clay has not undergone a substantial decrease from 1956 to 1993.

The void ratio after 100 percent consolidation was estimated using a range of values for the compression index (C_C). The range of C_C (0.58 to 1.36) was estimated using data from an empirical correlation and oedometer test results described in the next section. The void ratio after 100 percent consolidation was estimated at a particular depth using the following expression:

$$\Delta e = C_c * \log\left(\frac{\sigma_2'}{\sigma_1'}\right) \tag{10}$$

where Δe is the change in void ratio and σ' is the effective vertical stress. Therefore, the initial void ratio of the marine clay is 2.5 and the change in effective vertical stress is the difference between the current effective vertical effective stress and the effective stress after 100 percent consolidation is achieved.

Figure 24 illustrates the change in void ratio and bulk density that typically occurs in dredged material. This relationship between water content and void ratio was developed using a liquid limit of 71, a plasticity index of 41, and a specific gravity of soil mass equal to 2.71 (Table 1) for the Craney Island dredged material. Dredged material usually enters a management area from a discharge pipe at a void ratio of 10 to 20, or a bulk density of 1.15 to 1.08 kg/liter. With surface management, the decant water content can be reached at a void ratio of 4 to 5. Surface management includes removing weir boards to decant the surface water after sedimentation of the dredged material is complete. With additional surface management and some desiccation, the

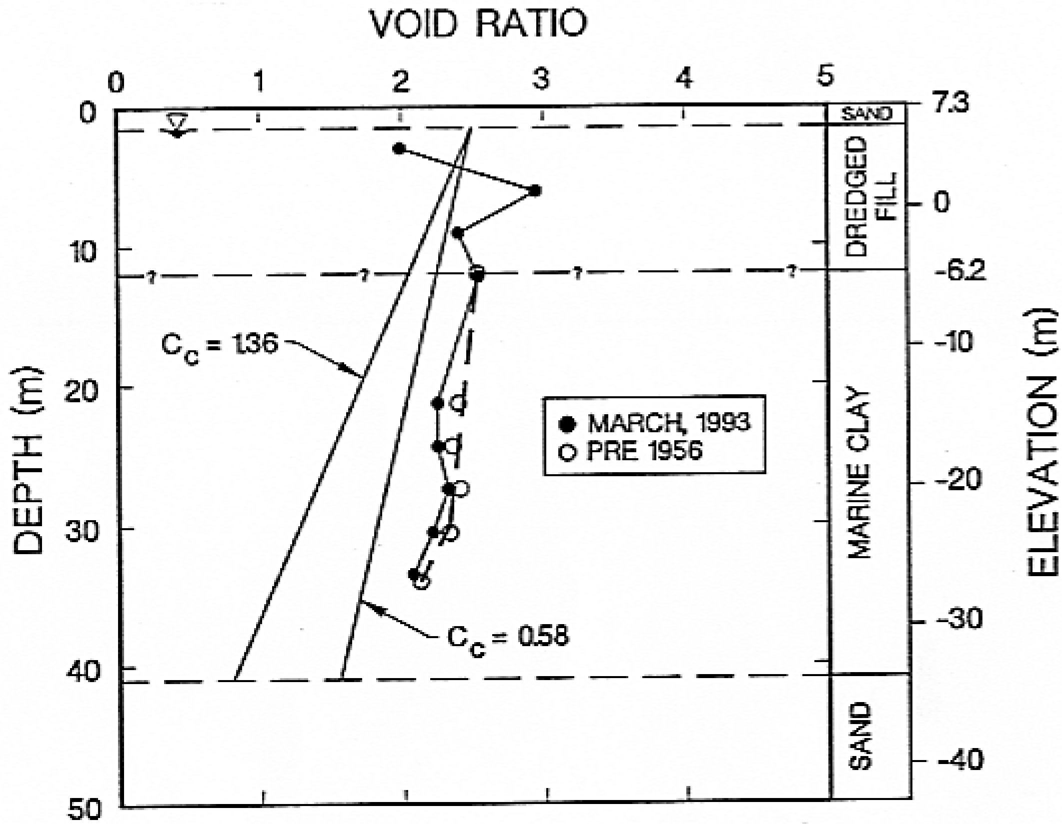

Figure 23. Predicted and Measured Void Ratio versus Depth in the Craney Island Strip Drain Test Section

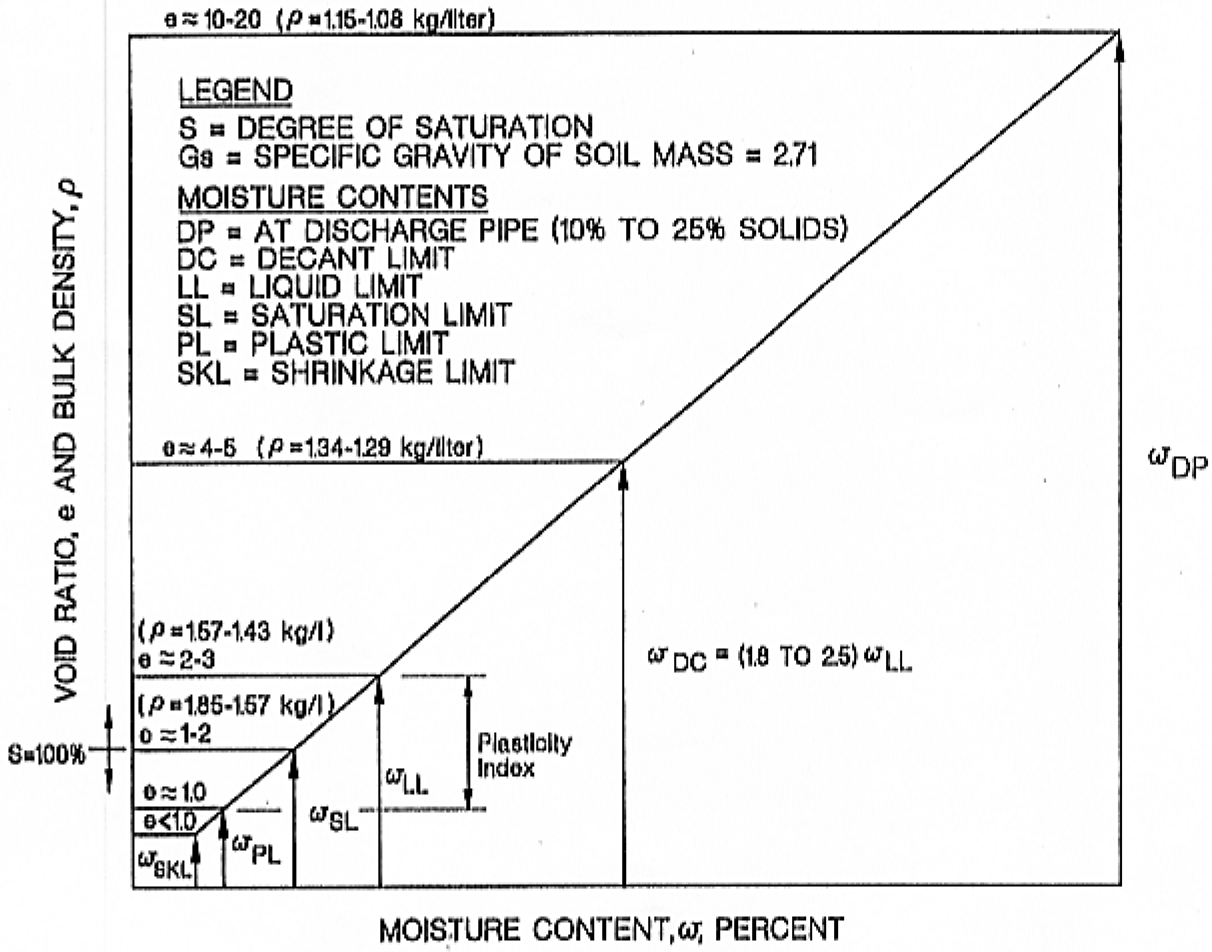

Figure 24. Moisture Content-Void Ratio Relationship for Craney Island Dredged Material

water content can be reduced to the liquid limit (void ratio of 2 to 3) and the saturation limit (void ratio of 1 to 2), respectively. Additional desiccation can reduce the degree of saturation below 100 percent and a water content corresponding to the plastic limit may be obtained. The lowest water content that can be obtained through desiccation is usually the shrinkage limit, which corresponds to a void ratio of less than 1.0.

From Figure 23 the void ratio of the dredged fill ranges from 2 to 3. It can be seen from Figure 24 that this void ratio corresponds approximately to the liquid limit. Therefore, the surface management program at the CIDMMA has been effective in reducing the void ratio to a water content that corresponds to approximately the liquid limit. However, additional decreases in void ratio could occur if consolidation is promoted.

Compression Index

Figure 25 presents a summary of oedometer tests reported in the General Design Memorandums (U.S. Army 1949). This data reflects the void ratio prior to construction of Craney Island. It can be seen that it is difficult to estimate a value of compression index (C_C) for the marine clay from this data. Ishibashi et al. (1993) suggested a value of C_C equal to 1.36 (Figure 25). Headquarters (1990) presents the following empirical correlation for clay of medium to low sensitivity:

$$C_c = 0.01 * (LL - 13\%) \tag{11}$$

This equation and a liquid limit of 71 were used to estimate a value of C_C equal to 0.58. It can be seen from Figure 25 that the range in C_C is large.

Two lines were drawn through the data using C_C equal to 0.58 and 1.36 (Figure 25). This was accomplished by estimating the change in void ratio for a particular change in effective stress using an initial void ratio of 3.0 and Equation (10). It can be seen that the expression for C_C in Equation (11) provides a reasonable approximation of the data. Clearly the value of C_C equal to 1.36 reported by Ishibashi et al. (1993) does not represent the majority of the void ratio-effective stress data in Figure 25. However, both values of C_C were used in a subsequent section to estimate the consolidation settlement induced by installation of strip drains. It should be noted that C_C ranges from 0.41 to 0.79 for the majority of the data with an average or representative value of 0.58.

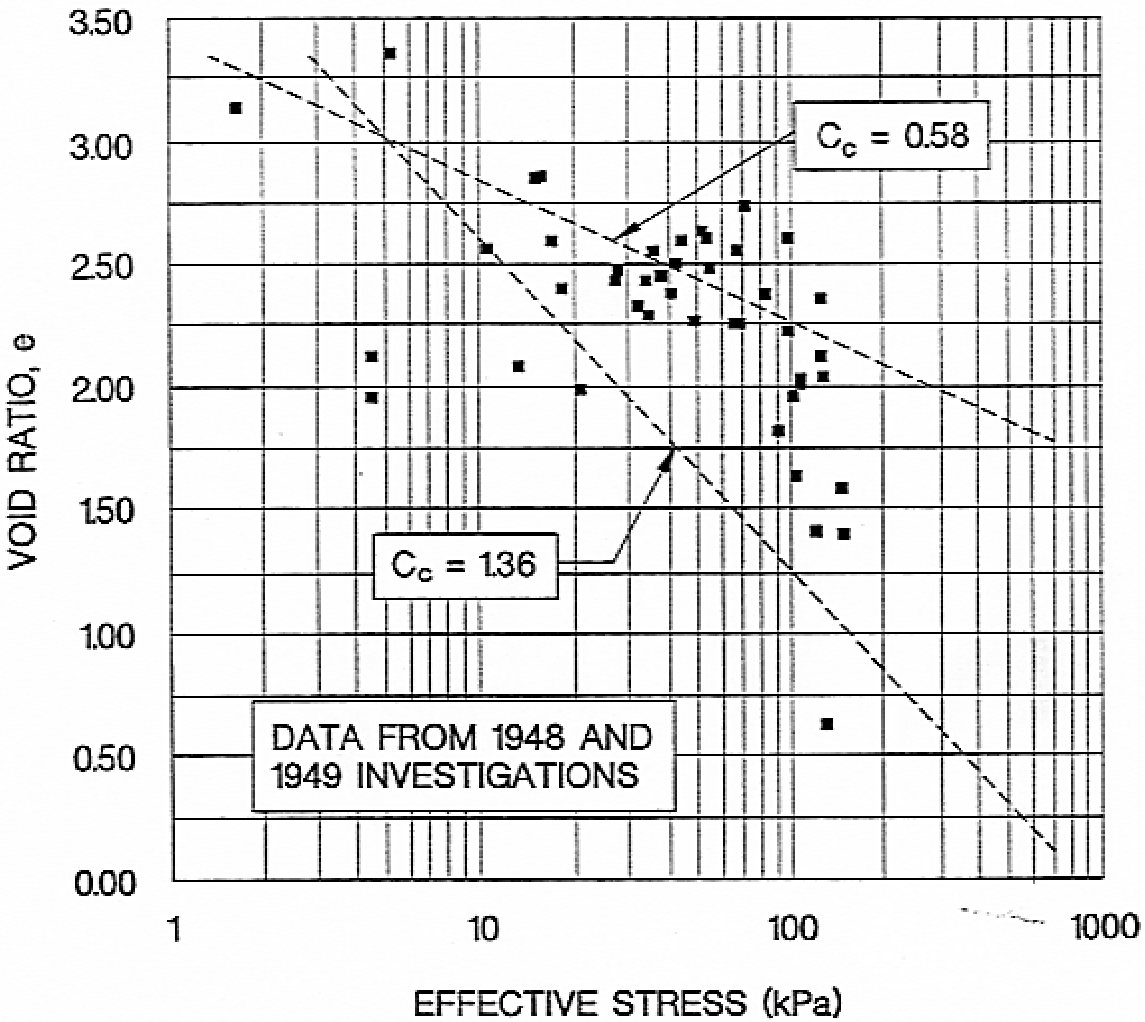

Figure 25. Void Ratio-Effective Stress Relationship for Marine Clay

Coefficient of Consolidation

Strip drain spacing is governed by the horizontal (C_h) and vertical (C_v) coefficients of consolidation. It can be seen from Figure 7 that strip drains will penetrate the dredged fill and marine clay which have different hydraulic conductivities. These soil types are similar, but the void ratio of the dredged fill is larger than the marine clay. This results in a higher hydraulic conductivity and coefficient of consolidation for the dredged fill than the marine clay. The results of the subsurface investigation were used to estimate design values of C_h and C_v for the dredged fill and marine clay.

The results of hydraulic conductivity tests in piezometers installed in the perimeter dikes were used to estimate C_h and C_v. In these tests, water in the piezometers is either pumped down or raised by filling. After pumping or filling is completed, the time required for the water level to return to the original or equilibrium condition is measured. The flow around the piezometer tip is probably a combination of vertical and horizontal flow. However, for simplicity the flow was assumed to be horizontal and thus the hydraulic conductivity tests were assumed to be measuring the horizontal hydraulic conductivity.

The value of horizontal permeability is calculated from these hydraulic conductivity tests using the following expression (British Standards Institution 1981):

$$K_h = \left[\frac{1.45 * A}{(t_2 - t_1)}\right]\log\left(\frac{H_1}{H_2}\right) \tag{12}$$

where K_h is horizontal hydraulic conductivity, t is time, A is the cross sectional area of the standpipe, and H is the variable hydraulic head at times t_2 and t_1. The values of C_h were calculated for the dredged fill using the horizontal hydraulic conductivity from Equation (12), an initial void ratio, e_o, of 3 (Figure 23), a unit weight of water equal to 9.8 kN/m^3, a horizontal coefficient of compressibility, a_h, of 9.1E-03 $(kPa)^{-1}$, and the following equation:

$$C_h = \left[\frac{K_h * (1 + e_0)}{a_h * \gamma_w}\right] \tag{13}$$

The value of a_h was obtained from oedometer and self weight consolidation test results (Cargill 1983) in the proper range of effective stress. The average horizontal hydraulic conductivity from three hydraulic conductivity tests in piezometers located in the dredged fill was estimated to be 2.4E-03 m/day. This hydraulic conductivity corresponds to an average value of C_h of 3.7E-02 m^2/day.

The values of C_h were calculated for the marine clay using the average horizontal hydraulic conductivity from field hydraulic conductivity tests in piezometers, an initial void ratio of 2.5 (Figure 23), and a coefficient of compressibility of 1.9E-02 $(kPa)^{-1}$ (Cargill 1983). The average horizontal hydraulic conductivity from three hydraulic conductivity tests in the marine clay was estimated to be 7.2E-04 m/day, which corresponds to an average C_h of 1.4E-02 m^2/day. As expected, the marine clay exhibited a lower horizontal hydraulic conductivity and C_h because of the smaller initial void ratio.

Installation of vertical strip drains, and thus disturbance, decreases the hydraulic conductivity of the soil, such that the ratio of the horizontal hydraulic conductivity to the vertical hydraulic conductivity ranges from 1.0 to 1.5 in marine clays (Mesri and Lo 1991). In undisturbed soil, this ratio can range from 3 to 10. Based on the data presented by Mesri and Lo (1991), a value of C_v was estimated by dividing C_h by an average ratio of 1.25. Therefore, the values of C_v for the dredged fill and marine clay were calculated to be 3.0E-02 and 1.1E-02 m^2/day, respectively (Table 3).

Table 3. Estimated Values of C_v and C_h for the Dredged Fill and Marine Clay

Source of Data	C_h (m^2/day)	C_v (m^2/day)
Dredge Fill Data		
Field Piezometers (1991)	3.70E-02	3.00E-02
Cargill (1983)	1.10E-02	8.80E-03
Marine Clay Data		
Field Piezometers (1991)	1.40E-02	1.10E-02
Design Memorandums (U.S. Army, 1949 & 1986)	1.90E-03	1.50E-03
Empirical Correlations (U.S. Navy, 1982)	9.90E-03	7.90E-03
Design Parameters	1.16E-02	9.29E-03

Oedometer test results from General Design Memorandums (U.S. Army 1949 and 1986) were also used to estimate C_v. Thirty-two time curves corresponding to the average effective stress in the marine clay (approximately 60 kPa) were used to estimate an average value of C_v for the test section area. The resulting average value of C_v for the marine clay is 1.5E-03 m^2/day. An average value of C_h was estimated to be 1.9E-03 m^2/day by multiplying C_v by 1.25.

It can be seen from Table 3 that the oedometer test results from the General Design Memorandums (U.S. Army 1949 and 1986) yield values of C_h and C_v that are lower than the field hydraulic conductivity test values. The difference is attributed to sample disturbance, the lack of a representative sample, the accuracy of evaluating hydraulic conductivity without flow quantity or pore-water pressure measurements in oedometer tests, and the combined vertical and horizontal flow that probably occurred around the piezometers during the field hydraulic conductivity tests.

Laboratory consolidation data on the dredged fill reported by Cargill (1983) were also used to obtain values of C_v and C_h equal to 8.8E-03 and 1.1E-02 m^2/day, respectively, for a void ratio of 3. Since these values of C_v and C_h were obtained from oedometer tests on the dredged fill at void ratios of approximately 3 (Cargill, 1983), these values are similar to the field hydraulic conductivity tests. The slight discrepancy may be related to soil disturbance, differences in soil type, combined vertical and horizontal flow around the field piezometers, and the presence of thin drainage layers around the piezometers.

Unfortunately, the piezocone dissipation test results are not suitable to estimate C_h and C_v for the dredged fill and marine clay. Dissipation tests were performed at various depths during cone penetration testing at three locations in the test section. The pushing of the cone through the soil creates shear induced excess pore-water pressures, which had not completely dissipated when the test was stopped. Theories relating dissipation time to C_h and C_v generally require the time required for 50 percent dissipation or consolidation. In the field, pore-water pressure versus time was plotted on an arithmetic scale and dissipation seemed complete. However, the end of primary consolidation, and thus 50 percent consolidation, could not accurately be determined when the results were plotted using a semi-logarithmic scale.

Empirical correlations of C_v presented in the Navy Design Manual DM-7.1 (U.S. Navy 1982) and a liquid limit of 71 were also used to estimate a value of C_v equal to 7.9E-03 m^2/day for the normally consolidated marine clay. This value of C_v corresponds to a value of C_h equal to 9.9E-03 m^2/day. Since the dredged fill is under going self-weight consolidation, values of C_v and C_h could not be estimated from this correlation. The values of C_v and C_h reported in this correlation correspond to effective stresses greater than those present in the dredged fill. Therefore, the dredged fill values of C_v and C_h are probably higher than those reported in the DM-7.1 correlation.

From Table 3 it can be seen that the values of C_h and C_v are uncertain. To facilitate the design of the test section it was decided to treat the dredged fill and marine clay as a single layer and use an average value of C_h and C_v. For design purposes, it was decided to use a weighted average value of C_h and C_v based on the thickness of the dredged fill and marine clay. The estimated average values of C_h and C_v are equal to 1.16E-02 and 9.29E-03 m^2/day, respectively, and were used to determine the preliminary spacing of the strip drains.

Strip Drain Design Parameters

The other major parameters required to develop an estimate of strip drain spacing are the well resistance and the extent of the smear zone. It can be seen from Figure 11 that the well resistance is governed by the ratio of K_h/K_w or K_h/q_w. Using field case histories, Lo (1991) showed that the effect of well resistance can be neglected if the parameter G is less than 0.2. Typical values of strip drain discharge capacity, q_w, range from 5.7 to 11.3 m^3/day (Koerner 1994). Since the consolidating clay is doubly drained, the maximum drainage length of the strip drain in the test section area (l_m) is equal to 22 m. This value of l_m also equals the maximum drainage length of vertical drainage path (H_{dr}) in the clay. Using these parameters, an average value of q_w equal to 8.5 m^3/day, and the average horizontal hydraulic conductivity measured in the field piezometers, the value of G ranges from 0.06 to 0.03. Therefore, well resistance may be neglected if the field discharge capacity of the strip drains is greater than 8.5 m^3/day.

The radial extent of the smear zone was studied using laboratory model tests by Onoue et al. (1991) and experience from pile driving and sand drain installations. This study revealed that the ratio of smear zone diameter to strip drain diameter, d_s/d_w, varies from 1.6 to 4.0. For design purposes the ratio of d_s/d_w was assumed to be 2. In addition, the horizontal hydraulic conductivity in the smear zone, K_s, was assumed to be one-half of the undisturbed hydraulic conductivity, K_h. This assumption is based on data presented by Onoue et al. (1991) that showed the ratio of K_s/K_h ranged from 0.2 to 1.0 in the smear zone.

Design of Test Section Strip Drains

Using the design theory presented by Lo (1991) and the design parameters previously described (Table 4), a sphere of influence of a strip drain, d_e, equal to 2.3 m is required to obtain a degree of consolidation of 90 percent in the dredged fill and foundation clay within one year. The value of d_e is obtained by an iterative process in which values of d_e are selected until Equation (5) yields a degree of consolidation of 90 percent.

The sphere influenced by each vertical strip drain is calculated using Equation (5) for both square and triangular drain patterns. Therefore, the sphere influenced by a vertical strip drain is the same for a square or triangular pattern. However, a triangular pattern provides better drainage for a specified area. The radius of influence does not reach the corners of a square area, and thus the square pattern will require a slightly longer time to consolidate than a triangular pattern. Since the sphere influenced by a square and triangular pattern is the same, it is recommended that a triangular pattern be used to facilitate drainage. A preliminary strip drain spacing for a triangular pattern was calculated to be 2.2 m by dividing d_e by 1.05 (Figures 10 and 11). The value of 1.05 is determined by geometry of three spheres creating a triangle.

The major design constraints for the test section were cost and the time required for 90 percent consolidation. Initially, it was decided that a consolidation time of 9 months was desired so that evaluation of the test section could be completed before specifications for construction of a subsequent strip drain contract would be required. To achieve 90 percent consolidation in nine months, a triangular pattern with a drain spacing of 1.8 m was recommended using the design parameters in Table 4. However, the lowest bid for the strip drain installation with a drain spacing of 1.8 m exceeded the project budget. As a result, the drain spacing was increased to 2.2 m to meet the project budget. This resulted in at least twelve to thirteen months being required to achieve 90 percent consolidation with a drain spacing of 2.2 m. Installation of the strip drains was completed in February, 1993, and thus 90 percent consolidation should have been achieved in February or March, 1994.

Table 4. Strip Drain Test Section Design Parameters

Parameter	Value
Degree of Consolidation	90%
Time	1 year
K_h	7.3E-04 m/day
C_v	9.3E-03 m^2/day
C_h	1.1E-02 m^2/day
H_{dr}	22 m
q_w	8.5 m^3/day
l_m	22 m
d_s/d_w	2.0
K_h/K_s	2.0

Strip Drain Installation Equipment

Vertical strip drains were installed in the test section using a novel piece of equipment. The equipment minimized disturbance to the sand blanket, confined dredged material, and the underlying marine clay during the installation operation. The equipment was developed by Geotechnics America, Inc. of Atlanta, Georgia (Figure 26). It can be seen that the 50 m high mandrel is stabilized using guy wires. The strip drain installation equipment had to be mounted on pontoons to reduce the maximum contact pressure to less than or equal to 10.4 kPa. This would enable the equipment to operate on the 15.2 to 30.5 cm thick desiccated crust in the mobility test section. The contractor mounted the installation equipment between two 2.1 m wide and 10.7 m long pontoons (Figure 27). The entire equipment weighs approximately 440 kN (99,120 lbs). Since the area of the pontoons is 45.6 m^2, the ground pressure exerted by this equipment is only 9.7 kPa. As a result, the equipment encountered little difficulty operating on the desiccated crust.

The drains were advanced using a mandrel sleeve that was pushed through the sand blanket, dredged material, marine clay, and into the underlying dense sand. The mandrel protects the drain material from tears, cuts, and abrasions during installation. The cross-sectional area of the mandrel was restricted to 6.5E-03 m^2 to reduce soil disturbance during installation. However, it should be noted that the mandrel is still considerably larger than the cross-sectional area of the strip (6.0E-04 m^2). The mandrel is retracted after each drain is installed at the required depth. A flat anchor plate is placed at the bottom of the mandrel to prevent soil from entering the bottom of the mandrel, to minimize tearing of the geotextile, and to anchor the drain material at the required depth when the mandrel is retracted.

The depth to the dense sand in the test section is approximately 40 to 50 m. The vertical strip drains had to be anchored in the dense sand underlying the marine clay to ensure that the drains would be doubly drained. To achieve this objective, each drain was pushed until a pressure of 69,000 kPa was applied to the mandrel. This was the maximum pressure that could be applied without lifting the equipment/pontoons off the ground surface.

The static method of installation with a constant rate of advancement was used for advancing the drains to reduce soil disturbance. An advancement rate less than 9.0 m per minute with the full static force was used. Approximately 0.1 m to 0.2 m of strip drain material was left protruding above the sand blanket in the main test section. In the mobility test section, a much longer length of strip drain was left protruding above the desiccated crust so that the vertical strip drain could be connected to the nearest horizontal strip drain. Horizontal strip drains were utilized in the mobility test section to evaluate their ability to convey water to the perimeter trenches. Horizontal strip drains were not used in the main area because the sand blanket provided drainage to the perimeter trenches. In addition, the sand blanket and horizontal strip

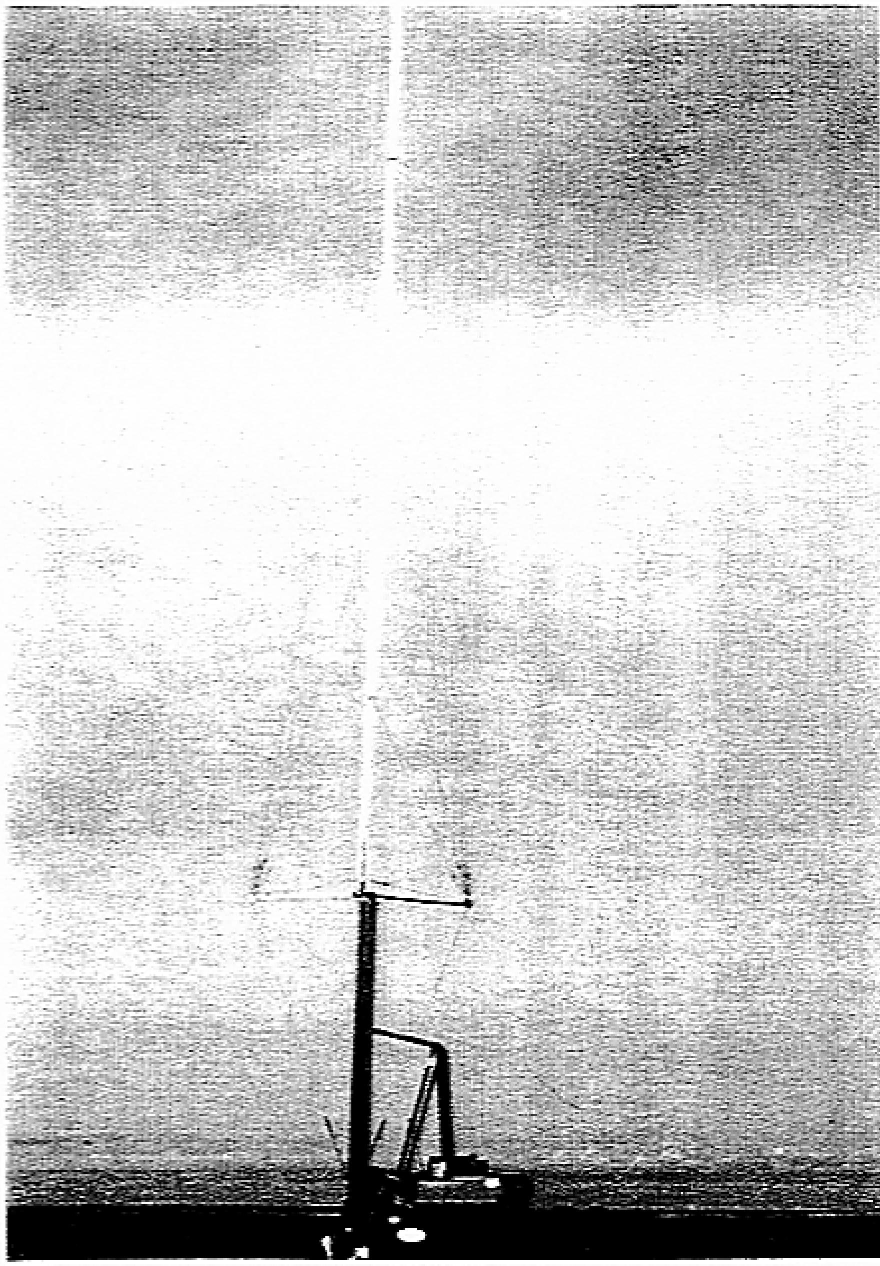

Figure 26. Overview of Strip Drain Equipment

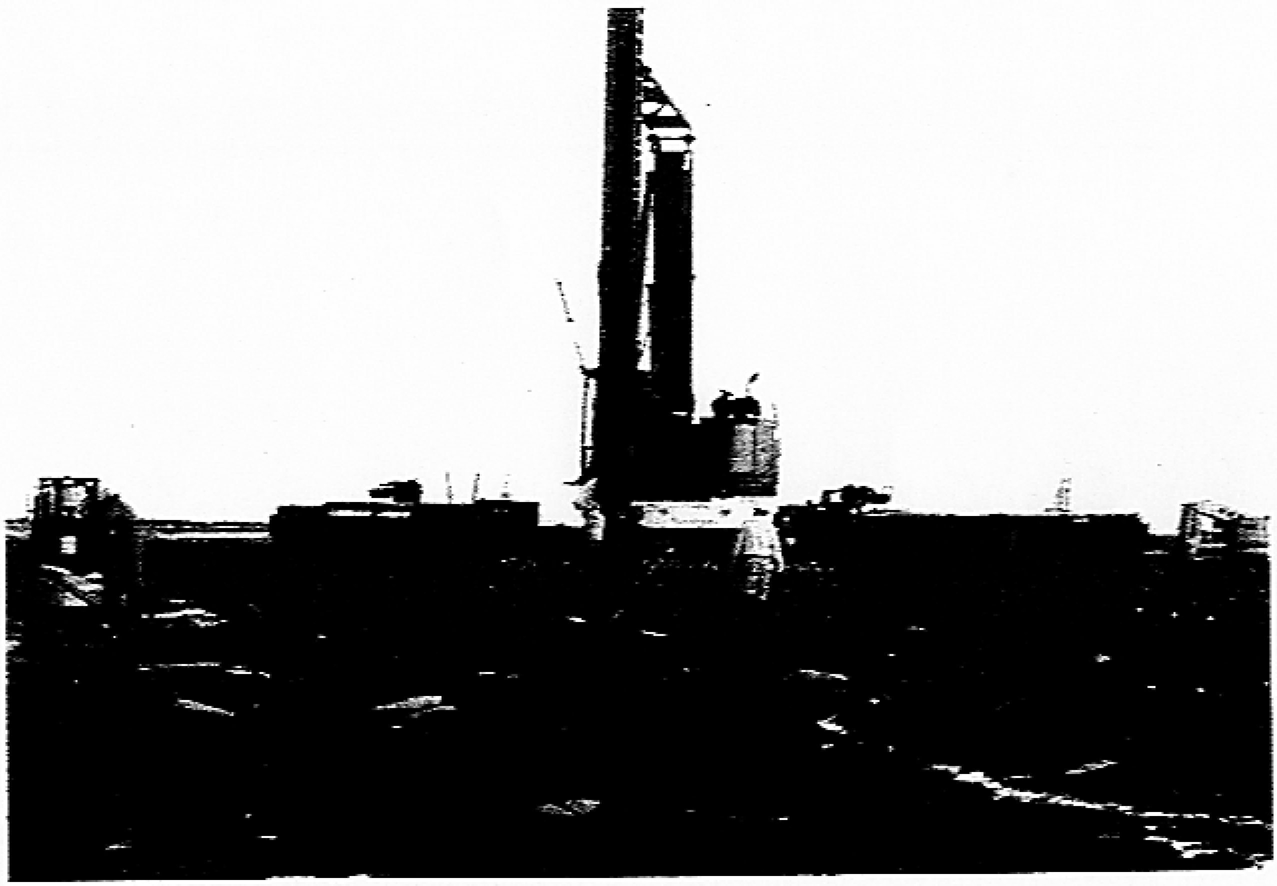

Figure 27. Installed Strip Drains and Pontoon Mounted Equipment

drains will provide an intermediate drainage layer as additional dredged material is placed in the area. The successful use of horizontal strip drains may preclude the cost and installation of a sand blanket throughout the remainder of the management area.

Strip drain installation in the test section began on 21 December 1992 and was completed on 26 February 1993. The total number of drains installed in the main and mobility test sections is 5,557. Approximately 193,824 lineal meters of vertical strip drain was installed in the main test section while 40,755 lineal meters of vertical strip drain was installed in the mobility test section. In the mobility section 2,181 lineal meters of horizontal strip drain were installed. The successful bid for the strip drain installation utilized the unit costs shown in Table 5.

Table 5. Strip Drain Costs for Craney Island Test Section

Item	Unit Price ($/m)	Quantity Installed (m)	Item Cost ($)
Vertical Strip Drains			
- Main Section	1.98	193,824	383,772
- Mobility Section	4.26	40,755	173,616
Horizontal Strip Drains			
- Mobility Section	49.18	2,181	107,262
		Total Project Cost:	$664,650

It can be seen that the unit pricing of the vertical strip drains in the mobility section is about two times higher than the main section. This was attributed to the uncertainty of operating the installation equipment on the desiccated crust. The low ground pressure of the installation equipment enabled the contractor to operate efficiently on the desiccated crust. As a result, it was concluded that a sand blanket is not required throughout the placement area to permit drain installation. This achieved one of the major objectives of the test section, which was to determine if strip drains could be installed without the use of a sand blanket. Based on the successful installation of strip drains in the mobility section, the unit price for installing vertical strip drains in a subsequent contract is only $1.98 per lineal meter (Joiner 1993). This is the same cost as the strip drains in the main test section. Therefore, the absence of a sand blanket will not affect the cost of future strip drain installation.

The performance of the strip drain test section is discussed in detail in the following chapter. However, the performance of the horizontal strip drains is discussed here for continuity purposes. In general, the horizontal strip drains conveyed water from the mobility section to the perimeter trench satisfactorily until the settlement exceeded approximately 1 m. Afterwards the large settlement bowl changed the direction of flow from the trench to the middle of the test section. As a result, the horizontal drains did not convey the expelled water to the perimeter trenches. If strip drains are installed throughout the placement area, it is recommended that horizontal strip drains not be used. The settlement bowl will cause a depression that will become filled with water. This depression will probably have to be drained using excavated trenches whether or not horizontal drains are installed.

Strip Drain Specifications

The vertical strip drains consist of a band-shaped plastic core enclosed in a suitable filter material. The polypropylene drainage core is wrapped in a filter made of a non-woven fabric of continuous filaments of 100% polypropylene. Strip drains with nipples or other individual protruding objects used to create a drainage channel were not specified for this project. It was anticipated that the lateral pressures in the marine clay are large and could cause the filter fabric to be punctured by protruding objects on the drainage core. The initial strip drain specifications also required the following physical characteristics (units as specified):

DRAIN:		
Weight	126 grams/meter (0.085 lbs/ft)	
Width	93 mm (3.7 inches)	
Thickness	4.1 mm (0.16 inch)	
Roll Length	305 m (1000 ft)	
Discharge Capacity	6.4×10^{-3} m^3/min (1.6 gpm)	ASTM D4716 (345 kN/m^2, 50psi)
Discharge Capacity	6.4×10^{-3} m^3/min (1.6 gpm)	ASTM D4716 (25% compression)
CORE:		
Material	Polypropylene	
Drainage Channels	54	
Grab Tensile Strength	21,390 kN/m^2 (3100 psi)	ASTM D1621/D638
Flex Modulus	1.1 kN/m^2 (0.16 psi)	
Density	0.90 g/cm^3 (56.2 pcf)	ASTM D792
One Side Wetted Perimeter	19.2 cm (7.56 in)	
FILTER FABRIC:		
Grab Tensile Strength	0.89 kN (200 lbs)	ASTM D4632
Grab Elongation at Break	60%	ASTM D4632
Modulus at 10% elongation	5.3 kN (1,200 lbs)	ASTM D1682/D4632
Trapezoidal Tear	0.33 kN (75 lbs)	ASTM D4533
Puncture Strength	0.31 kN (70 lbs)	ASTM D4833
Mullen Burst Strength	1373 kN/m^2 (210 psi)	ASTM D3786

Specific Gravity	0.95	
Permittivity	3238 lpm/m^2 (230 gpm/ft^2)	ASTM D4491
Permeability (K)	0.01 cm/min (0.0039 in/min)	ASTM D4491
Ultra Violet Resistance	70% at 500 Hours	ASTM D4355
Apparent Opening Size (AOS)	140	ASTM D4751

The applicable American Society for Testing and Materials (ASTM) standard test designations (Annual 1993) are presented in the specification. These specifications were developed to reduce the potential for the filter fabric to be squeezed into the channels of the drainage core. If the filter fabric was forced into the drainage channels, the discharge capacity of the drain would be significantly reduced. To reduce this possibility, a large number of drainage channels was specified (greater than or equal to 54) to reduce the area that the filter fabric had to span. A typical strip drain, for example, Amerdrain 407, has less than or equal to 40 drainage channels. In addition, a heavier filter fabric (186 g/m) was specified to resist the large lateral pressures in the marine clay. A heavier filter fabric also provides better flow characteristics. The thicker filter fabric reduces the amount of soil particles entering the drainage channels, which helps maintain the discharge capacity of the drain. The Amerdrain 410, manufactured by American Wick Drain Company, was proposed by the contractor to satisfy the specifications shown above.

In an effort to reduce the cost of the test section after the first bid, CENAO inserted the following strip drain specification:

DRAIN:		
Weight	0.80-1.30 N/Meter	
Width	90-105 mm	
Thickness	3-4 mm	
Discharge Capacity	60 mil/sec	ASTM D4716
GEOTEXTILE:		
Grab Tensile Strength	720 N	ASTM D4632
Puncture Strength	270 N	ASTM D4833
Water Permeability (K)	> 0.01 cm/sec	ASTM D4491

The main difference in the two specifications for vertical strip drains is that the second specification requires a plastic drainage core with only greater than 38 channels and a filter fabric weight of only 124 g/m. This resulted in the contractor installing the Amerdrain 407 manufactured by American Wick Drain Company.

The horizontal strip drains were also prefabricated with a polypropylene drainage core wrapped in a filter. The filter fabric is made of non-woven fabric of continuous filaments of 100 percent polypropylene. The contractor used a 0.1 m wide Akwadrain, manufactured by American Wick Drain Company, to satisfy the following specification for horizontal strip drains.

DRAIN:		
Weight	2400 g/m^2 (7.9 oz/ft^2)	
Width	305 mm (12 inches)	
Thickness	25 mm (1 inch)	
Compressive Strength	430 kN/m^2 (9000 psf)	ASTM D695/D1621
Shear Strength	430 kN/m (9000 lbs/ft)	ASTM D1621
Peel Strength	14.3 kN/m (35 lbs/ft)	ASTM D1876
Fungus Resistance	No Growth	ASTM G21
In Plane Discharge Capacity	0.0013 m^3/sec (20 gpm/ft width)	ASTM D4716 (Gradient = 0.1 at 10 psi)
CORE:		
Tensile Strength	0.6 kN (135 lbs)	ASTM D638
Fungus Resistance	No Growth	ASTM G21
FABRIC:		
Weight	1190 g/m^2 (3.9 oz/yd^2)	ASTM D3776
Grab Tensile Strength	0.5 kN (110 lbs)	ASTM D4632
Elongation at Break	40%	ASTM D4632
Modulus at 10% elongation	5.3 kN (1,200 lbs)	ASTM D1682/D4632
Trapezoidal Tear	0.22 kN (45 lbs)	ASTM D4533
Puncture Strength	0.33 kN (75 lbs)	ASTM D4833
Mullen Burst Strength	1518 kN/m^2 (220 psi)	ASTM D3786
Flow	1410 lpm/m^2 (100 gpm/ft^2)	ASTM D4491
Permeability (K)	0.15 cm/sec (0.059 in/sec)	ASTM D4491
Ultra Violet Resistance	80% at 500 Hours	ASTM D4355
Equivalent Opening Size	70	ASTM D4491
Fungus Resistance	No Growth	ASTM G21

4 TEST SECTION PERFORMANCE

Immediately following installation of the vertical strip drains, water could be seen rising along the drainage core and around the strip drain. The water rising around the strip drain was caused by the void left by the mandrel after retraction. Figure 28 is a photograph of a typical strip drain within 10 to 15 minutes after installation. It can be seen that water has risen 0.05 to 0.15 m above the ground surface inside the drain. A significant amount of water can also be seen rising around the drain. The cross-sectional area of the drain and mandrel are 6.0E-04 m^2 and 6.5E-03 m^2, respectively. This resulted in the creation of a void around the drain after the mandrel was withdrawn. This void usually closed shortly after installation because of the lateral earth pressures in the dredged fill and marine clay. Subsequently, flow only occurred inside the drain.

Immediate Settlement of Main Test Section

When a load is applied over a limited area on saturated clay, some settlement occurs immediately. This immediate settlement (S_i) occurs as a result of distortion, or change of shape of the clay beneath the loaded area. In other words, saturated clays undergo no immediate volume change because time is required for water to drain from the clay.

Immediate settlement can be estimated using elastic theory, by means of the following equation:

$$S_i = I_0 I_1 \frac{qB}{E}\left(1-\upsilon^2\right) \tag{14}$$

where I_0 and I_1 are dimensionless settlement factors presented by Christian et al. (1978), q is the average bearing pressure, B is the width of the loaded area, E is the undrained modulus of the clay, and υ is Poisson's ratio of the clay. The value of Poisson's ratio is equal to 0.5 for saturated clays.

The sand blanket was pumped into the north compartment using a dredge pipe from January, 1992 to early March, 1992. The first survey of the settlement plates in the sand blanket occurred on 18 December 1992. This is approximately nine months after placement of the sand blanket. As a result, it was assumed that most of the immediate settlement had occurred prior to the first survey of the settlement plates. Therefore, values of immediate settlement were not

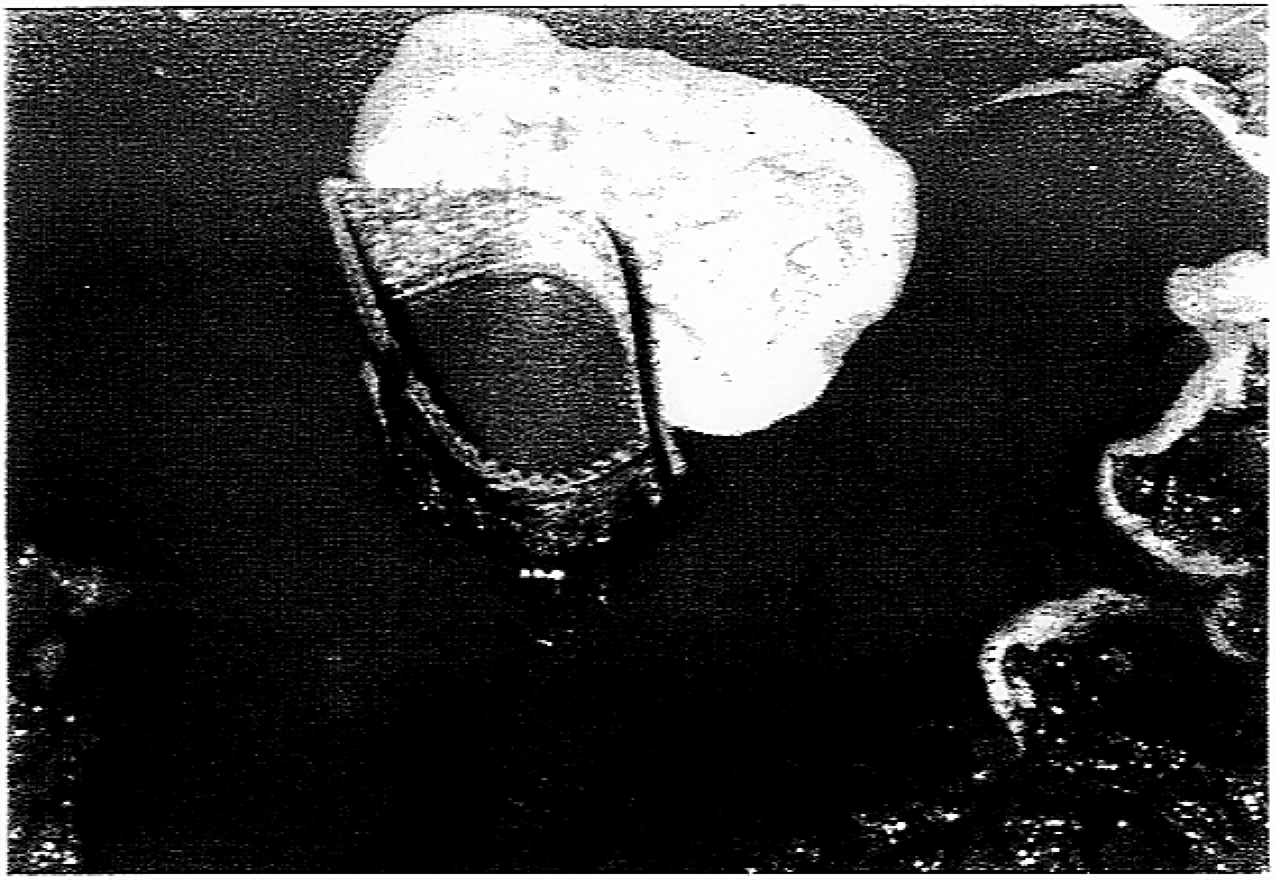

Figure 28. Water Rising in Strip Drain Immediately After Installation

included in estimating the total settlement that would be measured by the settlement plates in the test section.

Secondary Compression Settlement of Main Test Section

Secondary compression settlements can be estimated using the following equation:

$$S_s = C_\alpha * H_0 * \log\left(\frac{t}{t_p}\right) \tag{15}$$

where C_α is the coefficient of secondary compression, H_0 is the initial thickness of the layer undergoing secondary compression, t is the time at which secondary compression settlement is to be calculated, and t_p is the time at which primary consolidation ends and secondary compression begins. Mesri (1973) and Mesri et al. (1995) present empirical relationships for estimating the coefficient of secondary compression using the natural water content.

Initially, it was thought that the dredged fill and marine clay might undergo secondary compression settlement before new dredged material was pumped into the north compartment. It will be shown in a subsequent section that the consolidation had not reached 100 percent as of 18 July 1995 or approximately 25 months after strip drain installation. New dredged material was pumped into the north compartment from 21 August through December, 1995. As a result, the dredged fill and marine clay are undergoing additional primary consolidation because of the increase in vertical stress caused by the new dredged fill. As a result, secondary compression was not initiated at the test section.

In summary, secondary compression settlement was not added to the estimated consolidation settlements because the dredged fill and marine clay did not appear to reach the end-of-primary consolidation before new dredged material was placed in the north compartment.

Consolidation Settlement of Main Test Section

Several techniques were used to estimate the magnitude of consolidation settlement that would occur in the test section. First, consolidation settlement was calculated for only the marine clay by assuming instantaneous placement of the dredged fill at Craney Island. This case is unrealistic since it assumes instantaneous placement of the surcharge (dredged fill) and does not account for self-weight consolidation of the dredged fill or dissipation of excess pore-water pressures in the dredged fill. However, it does provide a lower bound estimate of the expected total settlement and an upper bound estimate of the marine clay settlement. The change in effective stress and Figure 25 were used to estimate the change in void ratio, and thus consolidation settlement using the following expression:

$$S = H_0\left(\frac{\Delta e}{1+e_0}\right) \tag{16}$$

where H_0 is the thickness of the consolidating layer, e_0 is the initial void ratio, and Δe is the change in void ratio. The final effective overburden stress was estimated using a unit weight of 15.4 kN/m^3 for the dredged fill and marine clay and a dredged fill surface at El. +7.3m CEMLW. This results in a 13.5 m and 30.4 m thick layers of dredged fill and marine clay, respectively. A unit weight of 15.4 kN/m^3 was estimated using an average void ratio after 100 percent consolidation of 2.0 and a degree of saturation of 100 percent. This value of unit weight, 15.4 kN/m^3 (98 pcf), is greater than the initial value of 14.6 kN/m^3 (93 pcf) reported by Ishibashi et al. (1993) because it reflects the densification caused by consolidation. For C_c equal to 0.58 the estimated settlement is 1.6 m, while for C_c equal to 1.36 the expected settlement is 4.1 m. As expected, this analysis method and $C_c = 0.58$ underestimates the measured settlement (2.3 to 2.7 m) because it does not include consolidation settlement of the dredged fill. However, it does indicate that the majority of settlement probably will occur in the marine clay.

Consolidation settlement estimates were also made using the undrained shear strength data obtained from cone penetration tests conducted prior to strip drain installation. The variation in S_u with depth (Figure 20) was determined from cone penetration tests using a value of N_k equal to 12. The current effective stress profile was estimated by dividing the values of S_u by an average value of S_u/σ_p' equal to 0.26. The difference between the current effective overburden stress distribution and the final effective stress distribution equals the increase in effective stress after 100 percent consolidation. The final effective overburden stress was estimated using a unit weight of 15.4 kN/m^3 for the dredged fill and marine clay and a dredged fill surface at El. +7.3 m CEMLW. Using values of C_c equal to 0.58 and 1.36, a range of consolidation settlement of 1.0 m to 2.3 m was estimated. Since settlement of the main test section has already exceeded 2.3 to 2.7 m in the main test section, this technique appears to underestimate the consolidation settlement. This is probably caused by the difficulties in estimating the current effective overburden stress profile, that is, the current excess pore-water pressure profile as previously discussed.

Consolidation settlement was also estimated using the change in void ratio caused by 100 percent consolidation and Equation (16). The current void ratio distribution (Figure 23) was determined from samples obtained from the boring at the center of the test section prior to strip drain installation. Void ratios corresponding to 100 percent consolidation were estimated using the final effective stress and the void ratio-effective stress relationships in Figure 25. The final effective overburden stress was estimated using a unit weight of 15.4 kN/m^3 for the dredged fill and marine clay and a dredged fill surface at El. +7.3 m CEMLW.

The change in void ratio caused by 100 percent consolidation was obtained from the difference between the March, 1993 void ratio profile (Figure 23) and the void ratio profile estimated using values of C_c. The dredged fill and marine clay were divided into sublayers and the change in void ratio for each sublayer was used to calculate the consolidation settlement. The estimated total consolidation settlement is the sum of the sublayer settlements. This technique is straight-forward and represents the best estimate of the field settlement. Consolidation settlements of 2.2 m and 6.1 m were estimated for the values of C_c equal to 0.58 and 1.36, respectively.

Estimated Settlement of Main Test Section

Consolidation settlements estimated using the previously described techniques are summarized in Table 6. In summary, it is estimated that the main test section will settle an average of approximately 2.2 m (C_c = 0.58) before 100 percent consolidation is achieved. The range of C_c in Figure 25 is 0.41 to 0.79 with a representative or average value being 0.58. This average value of C_c is also in agreement with the empirical correlation shown in Equation (11). The estimated consolidation settlement ranges from 1.5 to 3.0 m for values of C_c equal to 0.41 and 0.79, respectively, using the change in void ratio analysis method. Field settlements as of 18 July 1995 are 2.3 to 2.7 m in the main test section (Figure 29). Therefore, the average value of C_c (0.58) provides a reasonable estimate of the observed settlements.

Table 6. Estimates of Consolidation Settlement of Main Test Section

Analysis Method	C_c = 0.58 Settlement (m)	C_c = 1.36 Settlement (m)
Instantaneous Placement	1.6	4.1
S_u/σ_p' = 0.26 Analysis	1.0	2.3
Change in Void Ratio	2.2	6.1

If the main test section is assumed to have achieved a degree of consolidation of 90% as of 18 July 1995, and thus would exhibit a consolidation settlement of 2.7 m at 100% consolidation, a field value of C_c equal to 0.71 can be back-calculated from the observed settlements. This value of C_c is in agreement with the empirical relationship in Equation (11) and the void ratio-effective stress data presented in Figure 23. The back-calculated value of C_c was used to estimate a final consolidation settlement of approximately 2.9 m.

In summary, it is estimated that the main test section will settle between 2.7 and 2.9 m, which indicates that the section will undergo some additional settlement. A value of C_c equal to 0.71 should be used to estimate the consolidation settlement of future strip drain installations at the CIDMMA.

Measured Settlements in Main Test Section

Settlement plate readings for the main test section are presented in Figure 29. Installation of the vertical strip drains in the test section was completed on 26 February 1993. As of 18 July 1995 the maximum consolidation settlement in the main test section ranges from 2.3 to 2.7m. As a result, the measured settlements are in agreement with the predicted value of 2.7 to 2.9 m. In addition, the measured settlements are within the predicted range (1.5 - 3.0 m) for values of C_c equal to 0.41 and 0.79.

Figure 30 presents the settlement plate data from the main test section using a semi-logarithmic scale. It can be seen that none of the settlement plates indicate that primary consolidation has been completed. Therefore, consolidation is continuing, but it appears that the estimated settlements are in agreement with field measurements. A final conclusion on the accuracy of the estimated settlements will probably not be known because new dredged material was pumped into the north compartment from 21 August through December 1995.

For design purposes, it can be assumed that the north compartment will settle between 2.7 and 2.9 m if a sand blanket and strip drains are installed and 100 percent consolidation is allowed to occur. If consolidation settlements are to be estimated for the center and south compartments or the perimeter dikes, it is recommended that the change in void ratio analysis method (Figure 23) and a value of C_c equal to 0.71. Water content versus depth, and thus void ratio versus depth, information should be obtained from a representative boring to estimate the initial void ratio. The void ratio after 100 percent consolidation can also be estimated using a range of C_c

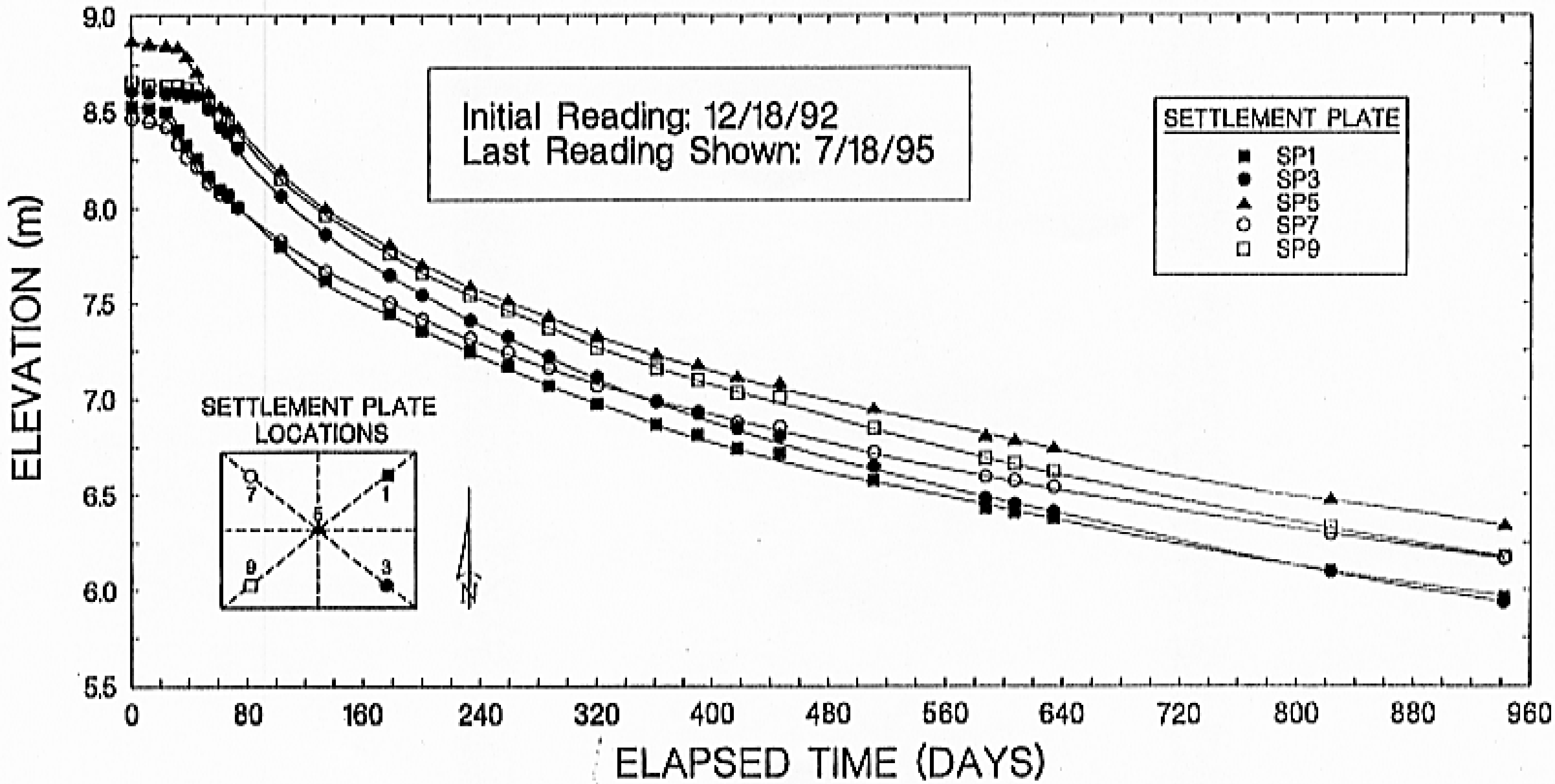

Figure 29. Settlement Plate Measurements in Main Test Section

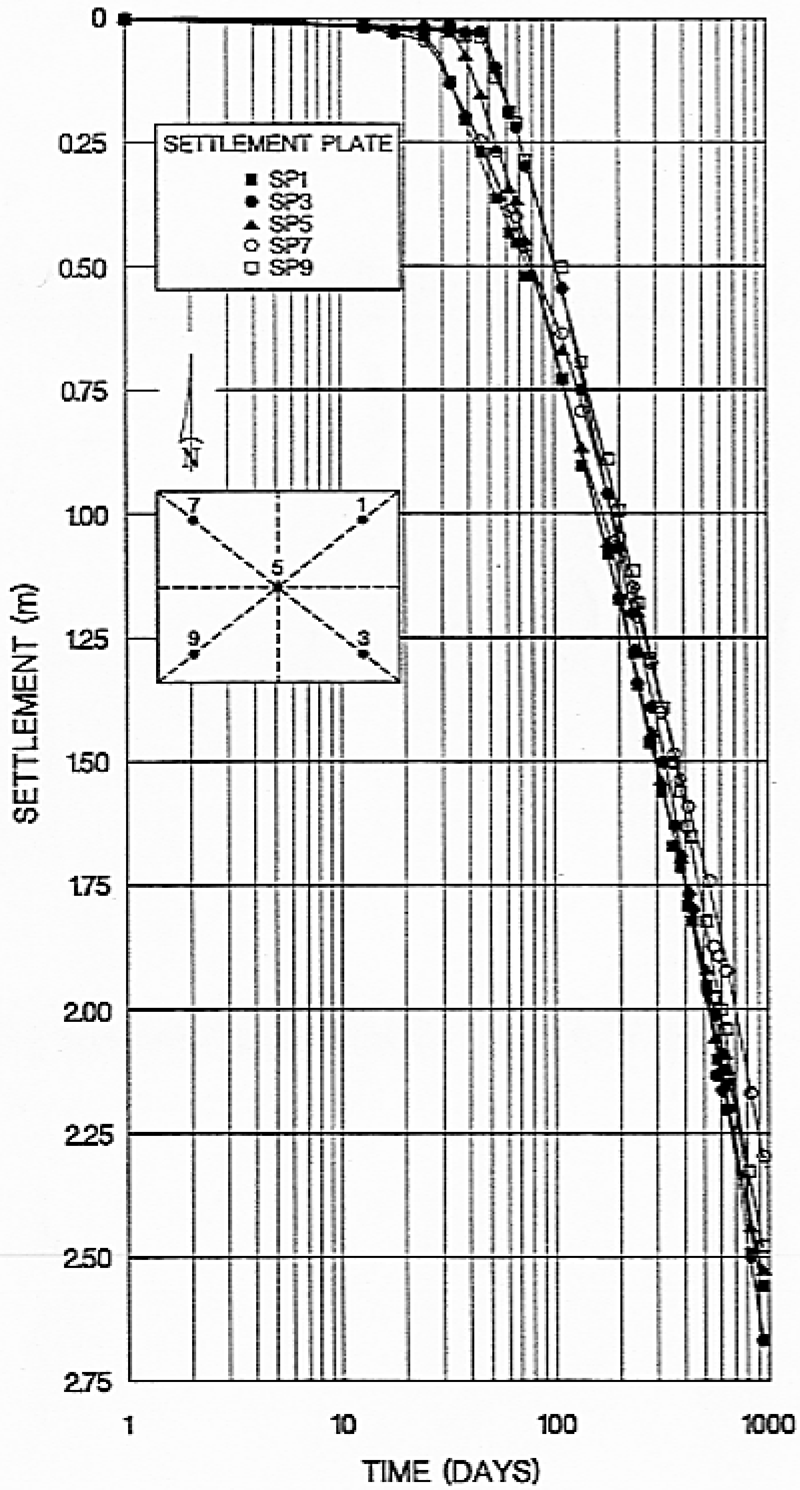

Figure 30. Semi-Logarithmic Presentation of Settlement Plate Measurements in Main Test Section

(0.41 to 0.79) and the final effective overburden stress to predict a range of consolidation settlement.

It should be noted that strip drains were installed in the northern part of the test area first. As a result, the settlement plates in the northern portion of the main test section (SP-1 and SP-7) show a faster response than the other settlement plates. For example, settlement plates SP-1 and SP-7 show a significant decrease in elevation after only 20 to 25 days. Conversely, settlement plates SP-3 and SP-9 did not show a significant decrease in elevation until 40 to 50 days after strip drain installation commenced.

Estimated and Measured Mobility Test Section Settlements

The mobility section was developed to demonstrate that a sand blanket was not required to support the strip drain equipment. Since the equipment exerted a ground pressure of only 9.7 kPa and was able to operate with little or no difficulty on the dredged material surface, a sand blanket is not required for future strip drain installations. A comparison of Figures 29 and 31 provides an insight into the effect of the sand blanket on the consolidation settlement of the dredged fill and marine clay. It can be seen that settlement plate SP-10 is located at the northern end of the adjacent mobility section and can be compared with settlement plates SP-1 and SP-7 at the northern end of the main section. Settlement plates SP-1 and SP-7 have settled 2.5 m to 2.3 m, respectively, while settlement plate SP-10 has settled only 1.85 m. Therefore, it may be concluded that the additional surcharge provided by the sand blanket results in a significant increase in consolidation settlement (0.45 to 0.65 m). It is anticipated that the additional consolidation primarily occurred in the dredged fill because of the compressible nature of the dredged material and the limited extent of the sand blanket.

In summary, the storage capacity lost by the installation of a sand blanket can probably be recouped by the subsequent consolidation of the underlying dredged fill. However, the cost of the sand blanket and the ability of the strip drain equipment to operate without the sand blanket will probably preclude the use of a sand blanket throughout the remainder of the placement area.

Figure 32 presents the settlement plate data from the mobility test section using a semi-logarithmic scale. It can be seen that none of the settlement plates indicate that primary consolidation has been completed and the measured settlements range from 1.75 to approximately 1.85 m. Table 7 presents the consolidation settlements estimated using the three previously described techniques. It is anticipated that the mobility test section will settle an average of approximately 1.7 m ($C_c = 0.58$) before 100 percent consolidation is achieved based on the change in void ratio analysis method. The estimated consolidation settlement ranges from 0.9 to

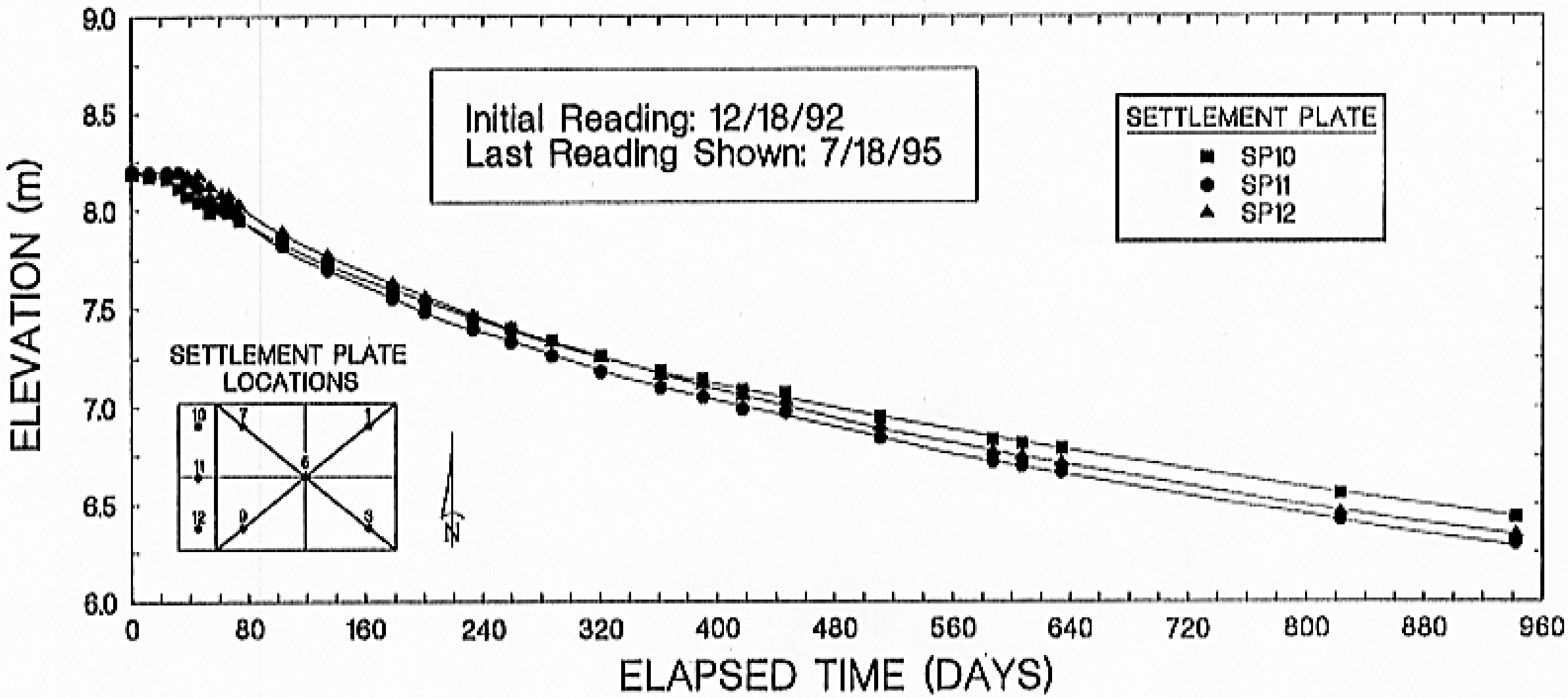

Figure 31. Settlement Plate Measurements in Mobility Test Section

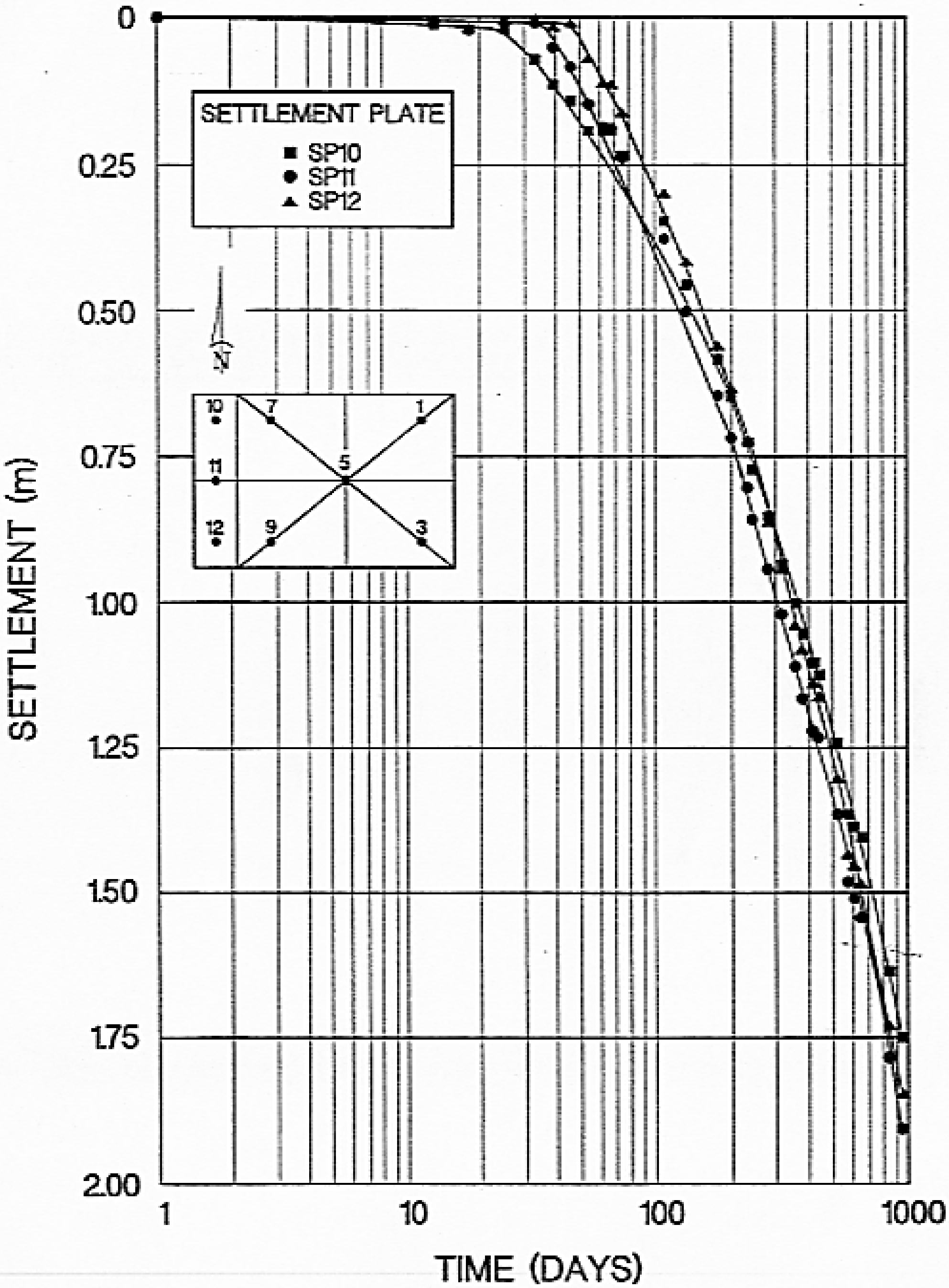

Figure 32. Semi-Logarithmic Presentation of Settlement Plate Measurements in Mobility Test Section

2.4 m for values of C_c equal to 0.41 and 0.79, respectively, and the change in void ratio analysis method.

The estimated consolidation settlement using the back-calculated value of C_c equal to 0.71 and the change in void ratio technique is approximately 2.3 m.

Table 7. Estimates of Consolidation Settlement of Mobility Test Section

Analysis Method	$C_c = 0.58$ Settlement (m)	$C_c = 1.36$ Settlement (m)
Instantaneous Placement	1.4	3.6
$S_u/\sigma_p' = 0.26$ Analysis	0.9	2.1
Change in Void Ratio	1.7	5.6

In summary, the mobility test section will probably settle between 2.1 and 2.3 m, which indicates that this section will undergo some additional settlement. A final conclusion on the accuracy of the estimated settlement will probably not be known because new dredged material was pumped into the north compartment during the 1995 summer and fall months. However, it can be assumed that the north compartment will settle between 2.1 to 2.3 m without a sand blanket after strip drains are installed. This information can be used along with the performance of the strip drains that are to be installed in the perimeter dikes, to determine whether strip drains should be installed in the placement area, the perimeter dikes, or both to extend the service life of the CIDMMA.

Time Rate of Consolidation

A strip drain spacing of 2.2 m should have resulted in 90% consolidation in twelve to thirteen months based on the strip drain design parameters in Table 4. Since the consolidation settlement was still occurring 25 months after drain installation, one or more of the design parameters does not model the field conditions. Several possible explanations for this discrepancy are:

1.) The Amerdrain 407, which was substituted for the specified Amerdrain 410 (see Part 3), did not exhibit a field discharge capacity of greater than or equal to 8.5 m^3/day. It is recommended that at least one strip drain be excavated from the test section and tested in a special triaxial apparatus at the Waterways Experiment Station to measure the discharge capacity. This information will aid interpretation of the test section results and quantify the field performance of the less expensive Amerdrain 407 for future projects.

2.) The mandrel insertion created a larger smear zone. It should be noted again that the cross-sectional area of the strip drain and mandrel are 6.0E-04 m^2 and 6.5E-03 m^2, respectively. The ratio of smear zone diameter (d_s) to strip drain diameter (d_w) usually varies from 1.6 to 4.0. For design purposes this ratio was assumed to be 2.0. Piezocone dissipation tests can be conducted radially from several strip drains to measure the changes in hydraulic conductivity radially from the drain. This information will clarify the d_s/d_w ratio mobilized in the field, aid interpretation of the test section results, and provide information for future projects that will likely involve this equipment.

3.) The strip drain is not acting as doubly drained. This could be caused by the drain not being anchored into the underlying sand or the large earth pressure at a depth of approximately 50 m significantly reducing the discharge capacity of the drain. If the drain is not doubly drained, the time required for 90% consolidation will approximately quadruple. If so, at least three to four years would be required to achieve 90% consolidation.

4.) The horizontal coefficient of consolidation is lower than 1.16E-02 m^2/day. This could be caused by a larger smear zone and/or variability in the field piezometer data, oedometer test results in the Design Memorandums, or the empirical correlation presented in the Navy Design Manual DM-7.1 (U.S. Navy 1982).

Figure 33 presents the measured and estimated consolidation settlement versus time for the main test section. The estimated relationships were obtained using the design parameters in Table 4 and the strip drain theory presented in Figure 11. The range in time rate of settlement was estimated using the degree of consolidation calculated using the procedure in Figure 11 and the final consolidation settlements (1.5 and 3.0 m) that correspond to C_c equal to 0.41 and 0.79, respectively. The measured settlements correspond to settlement plate SP-5, which is located at the center of the main test section. It was decided that the center of the main test section and the accompanying measured settlements (Figure 29) are representative of the time rate of

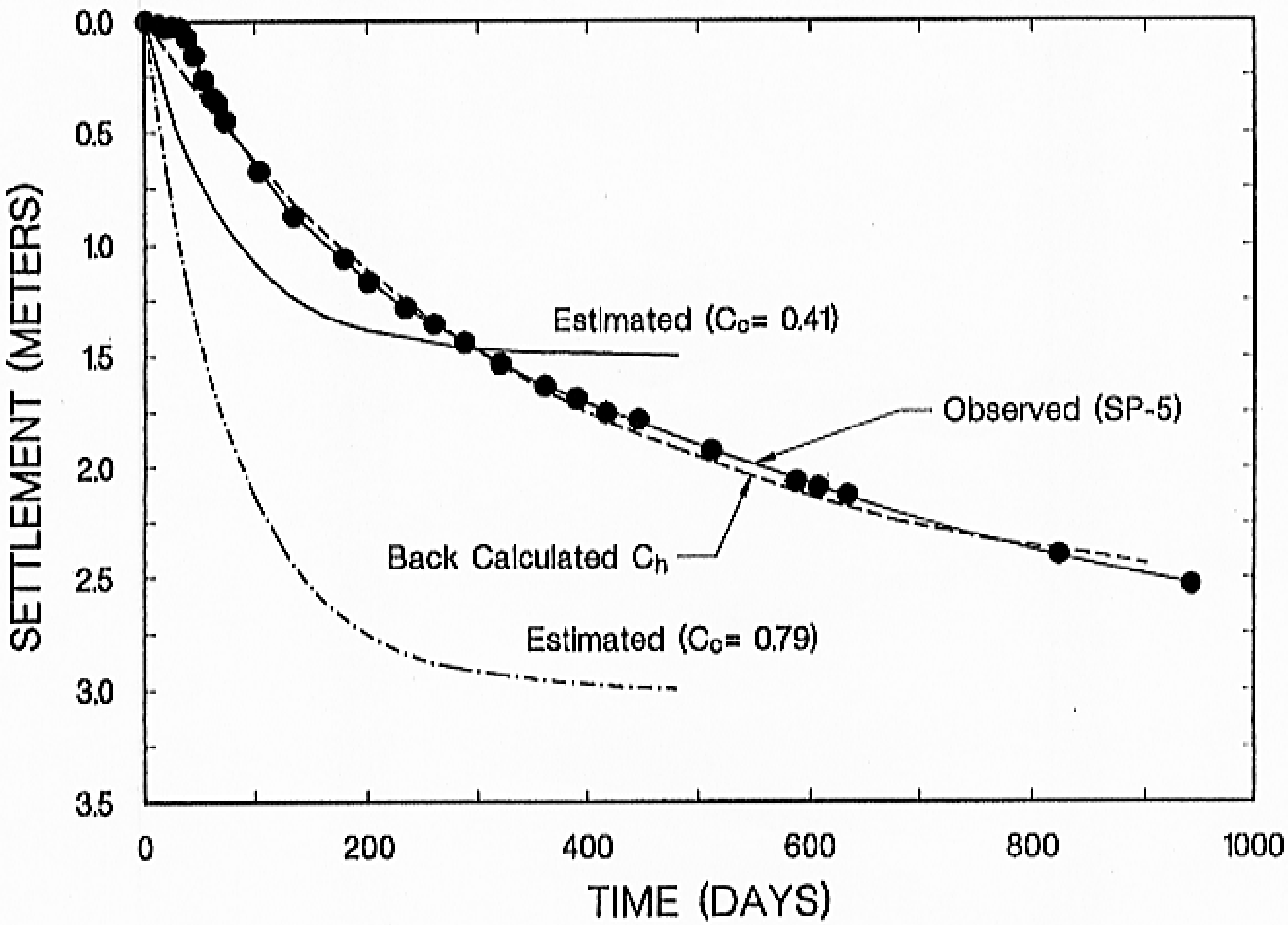

Figure 33. Measured and Estimated Time Rate of Consolidation Settlement for Main Test Section

consolidation of the main test section, and were thus compared to the estimated rates of consolidation.

It can be seen in Figure 33 that the estimated time rates of settlement are not in agreement with the measured values. This was expected because the strip drain spacing of 2.2 m was designed to achieve 90 percent consolidation in twelve to thirteen months and the test section was still settling 25 months after strip drain installation.

Figure 34 presents the measured and estimated consolidation settlement versus time for the mobility test section. The estimated relationships were obtained using the design parameters in Table 4 and the strip drain theory presented in Figure 11. The measured settlements correspond to settlement plate SP-11, which is located at the center of the mobility test section. It can be seen that the estimated time rates of settlement are also not in agreement with the measured values in the mobility test section.

The measured time rate of settlements in Figures 33 and 34 were used to back-calculate strip drain design parameters to aid future time rate of consolidation predictions. Figure 11 shows that the degree of consolidation for radial flow (Equation 5) depends on a number of parameters. A parametric study revealed that the degree of consolidation is significantly influenced by the value of C_h. As a result, the parameters in Table 4 were used to back-calculate the mobilized or field value of C_h using the theory in Figure 11. A mobilized value of C_h equal to 1.3E-03 m^2/day was calculated. This value is significantly lower than the design value of 1.1E-02 m^2/day (Table 4). This helps to explain why the test section did not reach 100 percent consolidation after twelve to thirteen months as designed. ,0013 .0085

In summary, it is recommended that a value of C_h equal to 1.3E-03 m^2/day be used for future strip drain design at the CIDMMA. However, it should be noted that this mobilized value of C_h reflects uncertainties in all of the design parameters in Table 4. In particular, this lower value of C_h probably reflects a number of uncertainties including drain discharge capacity, single versus double drainage, and extent of the smear zone. Therefore, this mobilized value of C_h represents the mobilized value for the strip drain equipment, installation procedure, and type of drain used in this test section. If the same or similar equipment, installation procedure, and strip drain are used, this value of C_h can be used for design purposes.

Excess Pore-Water Pressures

Figure 35 presents typical piezometric readings for the piezometers installed in the test section. Table 8 provides detailed information of the depth of each piezometer. Figure 35 presents the measurements from the piezometer cluster located at the center of the main test

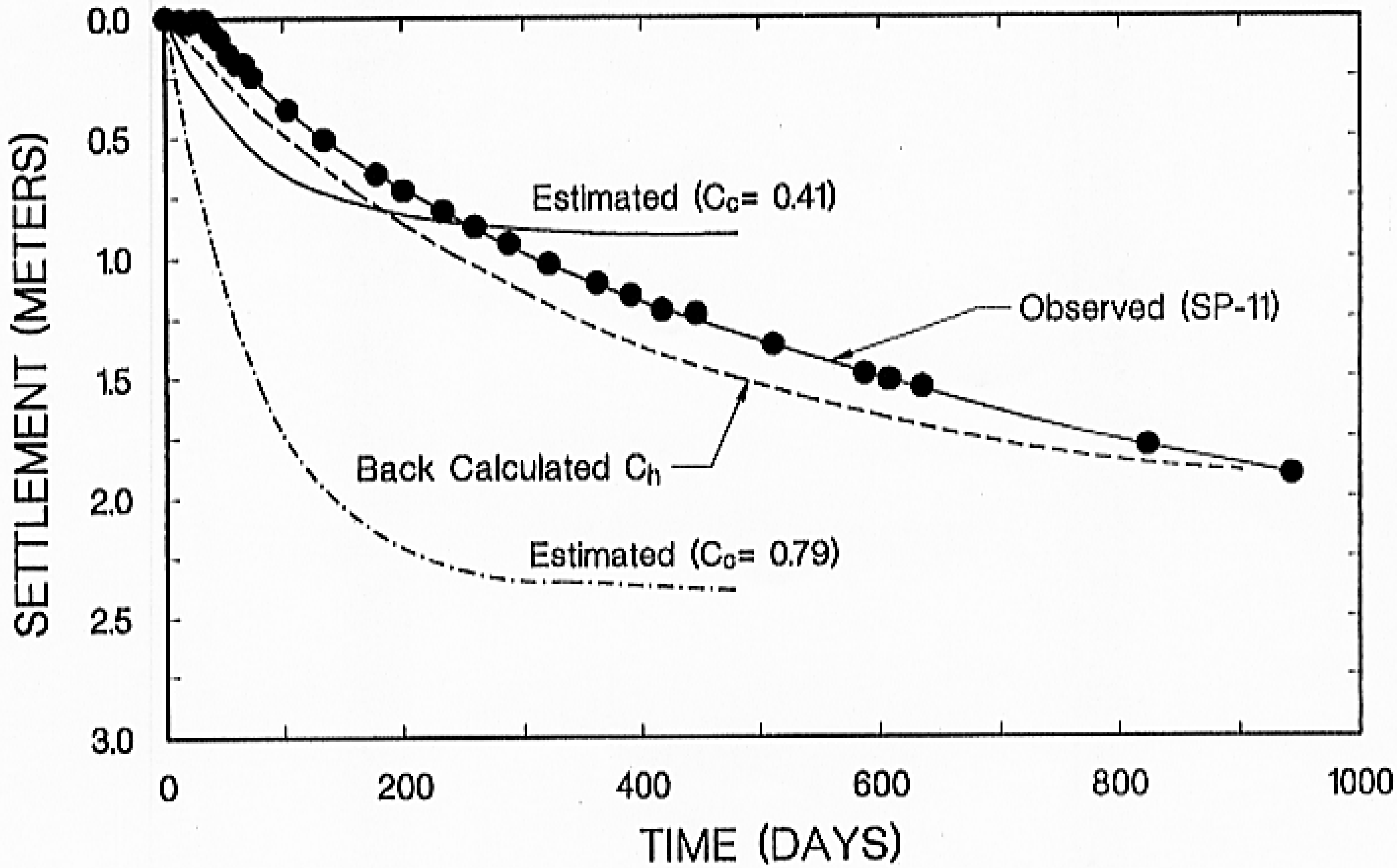

Figure 34. Measured and Estimated Time Rate of Consolidation Settlement for Mobility Test Section

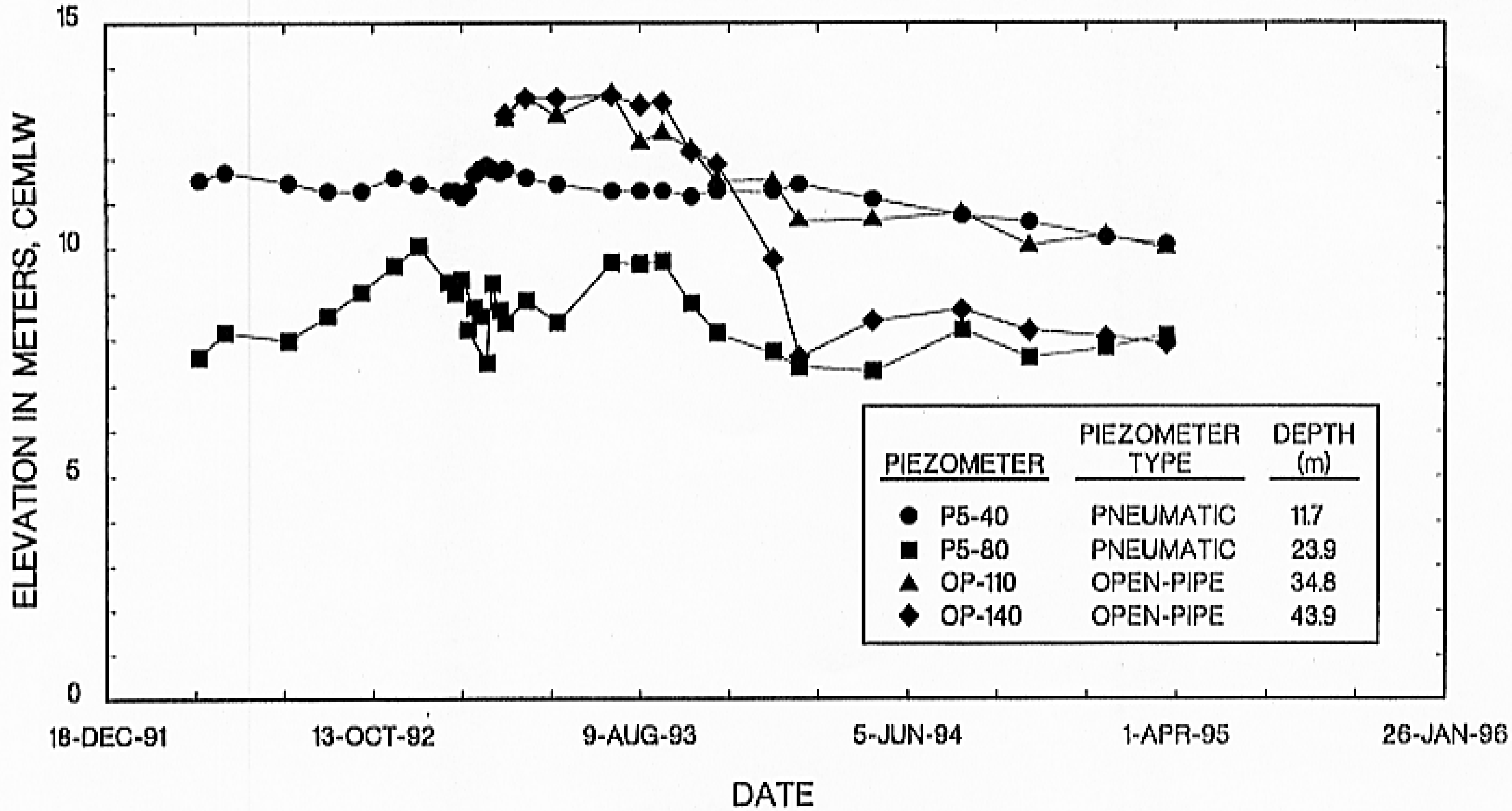

Figure 35. Typical Peizometer Measurements at Center of Main Test Section

section (P-5). The y-axis indicates the elevation of the water above the piezometer tip. For example, P5-80, with a tip elevation of -16.6 m and a reading of 8.2, exhibits a water height above the piezometer tip of 24.8 m.

Table 8. Detailed Piezometer Information

Piezometer	Type of Piezometer	Ground Elevation (m)	Piezometer Elevation (m)
P5-40	Pneumatic	+7.3	-4.4
P5-80	Pneumatic	+7.3	-16.6
OP-110	Open-pipe	+9.1	-25.7
OP-140	Open-pipe	+9.1	-34.9

It can be seen from Figure 35 that the pneumatic piezometers do not show a significant decrease in pore-water pressure that reflects the measured consolidation settlements. The open standpipe at a depth of 43.9 m shows a sharp decrease in hydraulic head in February, 1994. This open-pipe is located near the surface of the underlying dense sand and probably reflects the high hydraulic conductivity of the dense sand. The open-pipe at a depth of 34.8 m shows a more gradual decline in hydraulic head with time, which may reflect some of the observed consolidation.

The piezometer data, in general, does not correspond to the measured consolidation settlements. This trend has been noted by other researchers including (Hansbo et al. 1982). Hansbo et al. (1982) also showed that an increase in undrained shear strength was observed in several case histories with a negligible change in excess pore-water pressure. Mesri and Choi (1979) showed that when the effective vertical stress approaches the preconsolidation pressure, settlement continues at a nearly constant value of excess pore-water pressure. In other words, near the preconsolidation pressure settlement takes place at an almost constant value of effective vertical stress with a significant amount of excess pore-water pressure still remaining in the clay. The dredged material and marine clay are under- or normally-consolidated so the effective vertical stress is the preconsolidation pressure. This is probably the cause of the small decrease observed in the piezometric data.

Based on these results and the data presented by Mesri and Choi (1979), it is recommended that subsequent strip drain test sections in dredged material and normally consolidated clay rely more on settlement plate measurements, settlement points with depth, changes in water content or void ratio, and/or changes in cone penetration resistance than on piezometers to evaluate the effectiveness of strip drains. However, the cone penetrometer must be able to measure small changes in tip resistance to illustrate small increases in tip resistance.

Undrained Shear Strength

The undrained shear strength profile at 100 percent consolidation was estimated using an undrained strength ratio of 0.25 to 0.27. The current and estimated final values of S_u are presented in Figure 20. It can be seen that a substantial increase (85 to 90 percent) in S_u is predicted for portions of the marine clay.

A smaller increase is estimated for the dredged fill because this soil is not significantly under-consolidated. The presence of sand and silt seams in the dredged fill has allowed the excess pore-water pressures induced by self-weight consolidation to dissipate. As a result, there probably will not be a large increase in S_u in the dredged fill after strip drain installation. Cone penetration and field vane shear tests should be conducted in the near future to determine the increase in S_u in the dredged fill and marine clay. Again, the cone penetrometer should be able to measure small changes in tip resistance to demonstrate small increases in undrained shear strength.

Post Consolidation Subsurface Investigation

Cone and piezocone penetration tests should be conducted within 6 m of the previous cone penetration test locations. In addition, a boring should be drilled within 6 m of the previous borehole to measure the new water content and profiles. The change in water content and penetration resistance will be related to the increase in S_u and the magnitude of settlement. Quantifying the magnitude of settlement and the increase in S_u will aid in determining whether installing strip drains in the three compartments of Craney Island is economically feasible.

5 SUMMARY

A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying marine clay, and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since drains had never been installed in an active dredged material management area, a drain length of approximately 50 m was close to the longest drain ever installed, and the installation equipment had to operate directly on the surface of the soft dredged material.

It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future. Consolidation of the dredged fill and marine clay will also cause a large increase in soil shear strength. A supplemental study by the Principal Investigator will investigate the use of strip drains beneath exterior perimeter dikes to improve stability conditions. However, rip-rap and debris, long settled and submerged in the perimeter dikes, may restrict or prohibit installation of strip drains.

The subsurface investigation conducted prior to strip drain installation revealed that large excess pore-water pressures exist in the marine clay. The excess pore-water pressures in the dredged fill are smaller than the marine clay. The sand and silt seams in the dredged fill, identified from cone penetration test results, probably dissipate the excess pore-water pressures induced by the self-weight consolidation of the dredged fill. As a result, the large consolidation settlements that occurred at the test section are primarily attributed to consolidation of the marine clay.

In the main test section a sand working pad was constructed while no sand pad was constructed in the mobility test area. The main objective of the mobility test section was to determine if the installation equipment could operate directly on the soft dredged material. The pontoon mounted equipment was successful and efficient in operating in the mobility section. This significantly reduced the cost of vertical strip drains installed under a subsequent contract. Comparison of the settlements in the mobility and main test sections revealed that the sand blanket caused an increase in settlement by surcharging the dredged material. Therefore, surcharging confined dredged material can lead to substantial consolidation and increased in storage capacity. However, the cost of the sand blanket and the ability of the equipment to operate without the sand blanket will probably preclude the use of a sand blanket for future strip drain installations in the placement area.

Settlement plates installed in the main test section have settled approximately 2.3 to 2.7 m in 25 months. The mobility test section has settled 1.75 to 1.85 m in 25 months. These consolidation settlements are in agreement with the estimated values. The measured settlements were also used to estimate mobilized or field values of C_c and C_h. These mobilized values (C_c = 0.71 and C_h = 1.3E-03 m^2/day) should be used to design future strip drain installations at the CIDMMA that utilize similar equipment, installation procedure, and strip drain.

In summary, the Craney Island test section showed that strip drains are an effective technique for increasing the storage capacity, and thus service life, of confined dredged material management areas. This technique appears to be applicable to many management areas around the country.

Jamiolkowski, M., Ladd, C.C., Germaine, J.T., and Lancellotta, R. (1985). "New Developments in Field and Laboratory Testing of Soils," Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, California, Vol. 1, pp. 57-153.

Joiner, Russ. (1991). personal communication, President, Geotechnics America, Inc., Atlanta, GA, April.

Joiner, Russ. (1993). personal communication, President, Geotechnics America, Inc., Atlanta, GA, July.

Johnson, S.L. (1970). "Foundation Precompression with Vertical Sand Drains," Journal of Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 96, No. SM1, pp. 145-157.

Kjellman, W. (1948a). "Consolidation of Fine-Grained Soils by Prefabricated Drains," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 2, pp. 302-305.

Kjellman, W. (1948b). Discussion of "Consolidation of Fine-Grained Soils by Drain Wells," by R.A. Barron, Transactions. ASCE, Vol. 113, pp. 718-754.

Koerner, R.M. (1994). Designing with Geosynthetics. Third Edition, Prentice-Hall, Inc., Englewood Cliffs, NJ, pp. 783.

Koerner, R.M., Fowler, J., and Lawrence, L.A. (1986). "Soft Soil Stabilization Study for Wilmington Harbour South Dredged Material Disposal Area," Miscellaneous Paper GL-86-38, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 110 pp.

Koerner, R.M. and Fritzinger, S. (1988). "From Soft Soils to Heavy Construction," Civil Engineering Magazine, December, American Society of Civil Engineers, pp. 54-56.

Lo, D.O.K. (1991). "Soil Improvement by Vertical Drains," PhD Thesis, University of Illinois at Urbana-Champaign, IL, pp. 292.

Lunne, T. and Kleven, A. (1981). "Role of CPT in North Sea Foundation Engineering," Proceedings, Symposium on Cone Penetration Engineering Division, October, pp. 49-75.

Meigh, A.C. (1987). Cone Penetration Testing, Methods and Interpretation. CIRIA Ground Engineering Report: In-situ Testing, London, pp. 141.

Mesri, G. (1973). "Coefficient of Secondary Compression," Journal of Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 99, No. SM1, pp. 123-137.

Mesri, G. (1989). "A re-evaluation of $S_{u(mob)} = 0.22\ \sigma_p'$ using Laboratory Shear Tests," Canadian Geotechnical Journal, Vol. 26, pp. 162-164.

Mesri, G. and Choi, K. (1979). "Excess Pore Water Pressures During Consolidation," Proceedings, 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Singapore, Vol. 1, pp. 151-154.

Mesri, G. and Lo, D.O.K. (1991). "Field Performance of Prefabricated Vertical Drains," Proceedings, GEO-COAST '91 Conference, Yokahama, Japan, September, pp. 1-6.

Mesri, G., Stark, T.D., Aljouni, M., and Chen, C.S. (1996). "Secondary Compression of Peats," to be published in Journal of Geotechnical Engineering Division, ASCE, Vol. 122, No. GT11, pp.

REPORT DOCUMENTATION PAGE

1.

2. November, 1995

3. Final Report

4. Strip Drain Test Section in Craney Island Dredged Material Management Area

5.

6. Timothy D. Stark
Thomas A. Williamson

7. Department of Civil Engineering
University of Illinois at Urbana-Champaign
Urbana, IL 61801.

8. Technical Report
GL-95-XX

9. U.S. Army Corps of Engineers, Washington, DC 20314-1000

10.

11. Available from National Technical Information Service, 5285 Port Royal Road
Springfield, CA 22161

12.a. Approved for public release; distribution is unlimited.

13.

A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying foundation clay, and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since drains had never been installed in an active dredged material management area, a drain length of approximately 50 m was close to the longest drain ever installed, and the installation equipment had to operate directly on the surface of the soft dredged material. It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future. A supplemental study by the Principal Investigator will address the use of strip drains beneath exterior perimeter dikes to improve existing

stability conditions. Preliminary results show that the dredged fill and foundation clay are undergoing substantial (1.8 to 2.5 m in 25 months) consolidation settlement. In summary, prefabricated strip drains appear to be an economical technique for increasing the storage capacity of dredged material management areas.

14. Dredged Material
    Soil Densification
    Confined Dredged Material
    Prefabricated Strip Drains
    Service Life of Dredged Material Containment Facilities

15. XX

16.

17. UNCLASSIFIED

18. UNCLASSIFIED

19.

20.